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Utah Department of Transportation - Research Division
4501 South 2700 West, Box 148410 - Salt Lake City, Utah 84114-8410

Report No. UT-98.17

LIQUEFACTION HAZARD EVALUATION OF INTERSTATE, FEDERAL, AND STATE HIGHWAY BRIDGE SITES IN UTAH

Prepared For:

Utah Department of Transportation
Research Division

Submitted By:

Brigham Young University
Dept. of Civil & Environmental Engineering

Authored By:

Dr. T. Leslie Youd
P. Scot Willey
Samuel D. Gilstrap
Curt R. Peterson

December 30, 1998

UDOT RESEARCH DIVISION REPORT ABSTRACT

1. Report No. UT-98.17	**2. Government Accession No.**	**3. Recipient's Catalog No.**
4. Title and Subtitle LIQUEFACTION HAZARD EVALUATION OF INTERSTATE, FEDERAL, AND STATE HIGHWAY BRIDGE SITES IN UTAH	**5. Report Date** December 30, 1998 **6. Performing Organization Code**	
7. Author(s) T. Leslie Youd, P. Scot Willey, Samuel D. Gilstrap, Curt R. Peterson	**8. Performing Organization Report No.**	
9. Performing Organization Name and Address BYU, Department of Civil Engineering 368 Clyde Building Provo, UT 84602	**10. Work Unit No.** **11. Contract No**. UDOT – 93 – 7 – 8476 NCEER HIGHWAY PROJECT TASK No. 106-F-6.1	
12. Sponsoring Agency Name and Address Utah Department of Transportation Research Division 4501 South 2700 West Salt Lake City, UT 84119	**13. Type of Report and Period Covered** FINAL REPORT **14. Sponsoring Agency Code** PIC No. MP96.001	
15. Supplementary Notes		

16. Abstract

The purpose of this study is to evaluate the liquefaction hazard at bridge sites throughout the state of Utah. A secondary objective is to test and suggest revisions to the screening guide developed by Youd (1998) for the National Center for Earthquake Engineering Research (NCEER). The procedure involved regional screening of liquefaction potential, site-specific evaluations of liquefaction hazard, and calculation of liquefaction-induced ground failures. Due to the limited scope of this study, only an example calculation of ground failure is included.

The final objective of the study is to prioritize the bridge sites in Utah for future investigations. Sites are prioritized according to the presence of liquefiable sediment and the quality and availability of subsurface data. Sites classed as Priority I are located at river crossings and either have a confirmed presence of liquefiable sediment or insufficient information to evaluate liquefaction. Priority II sites have a confirmed presence of liquefiable sediment, but are not located at river crossings. Sites classed as Priority III are not located at river crossings and have insufficient data available to evaluate susceptibility. Priority IV sites have a confirmed low liquefaction hazard.

A principal emphasis of this study was evaluation of the I-15 corridor. The results show that nearly all of the bridge sites in Salt Lake County are underlain by possibly liquefiable sediment. An example calculation of liquefaction-induced ground failure hazard at the 600 South off ramp evaluated embankment stability and deformation, ground displacement due to lateral spread, ground settlement, and bearing capacity. The ground failure analyses showed that damaging ground displacements are not likely to occur at the 600 South site, even though liquefaction might occur.

A few hundred bridge sites were reviewed by regional screening procedures and 325 bridge sites were analyzed using the site-specific screening procedures. Of these sites, 279 were identified as underlain by possibly liquefiable soil layers. Twenty-five of these sites were identified as Priority I sites for further investigation (table 14). The Priority I sites are at river crossings, a setting in which most past bridge damage due to liquefaction has occurred. The remaining 254 bridge sites possibly underlain by liquefiable sediment were classed as Priority II sites. These sites are tabulated in appendix A.

A companion report, the "Executive Summary & Implementation Plan", UT-98.17A, summarizes the contents of this report and outlines a proposed two-phase implementation plan to study and remediate high priority sites.

17. Key Words Utah Bridges, Liquefaction, Earthquakes		**18. Distribution Statement** Utah Department of Transportation Research Division PO Box 148410 Salt Lake City, UT 84114-8410	
19. Security Classification (of this report) UNCLASSIFIED	**20. Security Classification (of this page)**	**21. No. of Pages** 147	**22. Price**

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Acceleration	$9.81\ m/s^2 = 386.22\ in./s^2 = 32.185\ ft/s^2$, Paris: $g = 9.80665\ m/s^2$ London: $g = 3.2174 \times 10^1\ ft/s^2$
Area	$1\ m^2 = 1.5500 \times 10^3\ in.^2 = 1.0764 \times 10^1\ ft^2 = 1.196\ yd^2 = 10^6\ mm^2 =$ $10^4\ cm^2 = 2.471 \times 10^{-4}\ acres^2 = 3.861 \times 10^{-7}\ mi^2$
Coefficient of Consolidation	$1\ m^2/s = 10^4\ cm^2/s = 6 \times 10^5\ cm^2/min = 3.6 \times 10^7\ cm^2/h$ $= 8.64 \times 10^8\ cm^2/day = 2.628 \times 10^{10}\ cm^2/month$ $= 3.1536 \times 10^{11}\ cm^2/year$ $= 1.550 \times 10^3\ in.^2/s = 4.0734 \times 10^9\ in.^2/month$ $= 1.3392 \times 10^8\ in.^2/day = 4.8881 \times 10^{10}\ in.^2/year$ $= 9.4783 \times 10^5\ ft^2/day = 2.8830 \times 10^7\ ft^2/month$ $= 3.3945 \times 10^8\ ft^2/year$
Flow	$1\ m^3/s = 10^6\ cm^3/s = 8.64 \times 10^4\ m^3/day = 8.64 \times 10^{10}\ cm^3/day$ $= 3.5314 \times 10^1\ ft^3/s = 3.0511 \times 10^6\ ft^3/day$
Force	$10\ kN = 2.2482 \times 10^3\ lb = 2.2482\ kip = 1.1241\ t$ (short ton = 2000 lb) $= 1.0194 \times 10^3\ kg = 1.0194 \times 10^6\ g = 1.0194\ T$ (metric ton = 1000 kg) $= 10^9$ dynes $= 3.5971 \times 10^4$ ounces $= 1.022$ tl (long ton = 2200 lb)
Force per Unit Length	$1\ kN/m = 6.8526 \times 10^1\ lb/ft = 6.8526 \times 10^{-2}\ kip/ft$ $= 3.4263 \times 10^{-2}\ t/ft$ $= 1.0194 \times 10^2\ kg/m = 1.0194 \times 10^{-1}\ T/m$
Length	$1\ m = 3.9370 \times 10^1\ in. = 3.2808\ ft = 1.0936\ yd$ $= 10^{10}$ Amgstrom $= 10^6$ microns $= 10^3\ mm = 10^2\ cm$ $= 10^{-3}\ km = 6.2137 \times 10^{-4}$ mile $= 5.3996 \times 10^{-4}$ nautical mile
Moment or Energy	$1\ kN.m = 7.3759 \times 10^2\ lb.ft = 7.3759 \times 10^{-1}\ kip.ft = 3.6879 \times 10^{-1}\ t.ft$ $= 1.0194 \times 10^3\ g.cm = 1.0194 \times 10^2\ kg.m = 1.0194 \times 10^{-1}\ T.m$ $= 10^3\ N.m = 10^3$ Joule
Moment of Inertia	$1\ m^4 = 2.4025 \times 10^6\ in.^4 = 1.1586 \times 10^2\ ft^4 = 6.9911 \times 10^{-1}\ yd^4$ $= 10^8\ cm^4 = 10^{12}\ mm^4$
Moment per Unit Length	$1\ kN.m/m = 2.2482 \times 10^2\ lb.ft/ft = 2.2482 \times 10^{-1}\ kip.ft/ft$ $= 1.1241 \times 10^{-1}\ t.ft/ft$ $= 1.0194 \times 10^2\ kg.m/m = 1.0194 \times 10^{-1}\ T.m/m$
Pressure	$100\ kPa = 10^2\ kN/m^2 = 1.4503 \times 10^1\ lb/in.^2 = 2.0885 \times 10^3\ lb/ft^2$ $= 1.4503 \times 10^{-2}\ kip/in.^2 = 2.0885\ kip/ft^2$ $= 1.0442\ t/ft^2 = 7.5003 \times 10^1$ cm of Hg (0 °C) $= 1.0197\ kg/cm^2 = 1.0197 \times 10^1\ T/m^2$ $= 9.8689 \times 10^{-1}$ Atm $= 3.3455 \times 10^1$ ft of H_2O (4 °C) $= 1.0000$ bar $= 10^6\ dynes/cm^2$
Temperature	°C = 5/9 (°F - 32), °K = °C + 273.15
Time	1 yr. = 12 mo. = 365 day = 8760 hr $= 5.256 \times 10^5$ min $= 3.1536 \times 10^7$ s
Unit Weight, Coefficient of Subgrade Reaction	$10\ kN/m^3 = 6.3654 \times 10^1\ lb/ft^3 = 3.6837 \times 10^{-2}\ lb/in.^3$ $= 1.0196\ g/cm^3 = 1.0196\ T/m^3 = 1.0196 \times 10^3\ kg/m^3$
Velocity or Permeability	$1\ m/s = 3.6\ km/h = 2.2369\ mile/h = 6 \times 10^1\ m/min = 10^2\ cm/s$ $= 1.9685 \times 10^2\ ft/min = 3.2808\ ft/s$ $= 1.0346 \times 10^8\ ft/year = 2.8346 \times 10^5\ ft/day$
Volume	$1\ m^3 = 6.1024 \times 10^4\ in.^3 = 3.5315 \times 10^1\ ft^3 = 7.6455\ 10^{-1}\ yd^3$ $= 10^9\ mm^3 = 10^6\ cm^3 = 10^3\ dm^3$ $= 10^3$ liter $= 2.1998 \times 10^2$ gallon (U.K.) $= 2.6417 \times 10^2$ gallon (U.S.)
Volume Loss in a Tubing	$1\ cm^3/m/kPa = 8.91 \times 10^{-4}\ in.^3/ft/psf$

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ACKNOWLEDGMENTS

The success of this report is due in large part to the many people and organizations who supported it.

Funding for this study was provided by the Utah Department of Transportation (UDOT) and the National Center for Earthquake Engineering Research (NCEER). Field and laboratory data was provided to Samuel Gilstrap by, Dames & Moore Inc., Kleinfelder Inc., Rollins Brown & Gunnell Inc., and Terracon Inc.

Steven Bartlett of Woodward Clyde developed the database program used for the analysis, which proved to be very helpful. His direction on the use of the program is greatly acknowledged.

CHAPTER 1 - INTRODUCTION

Liquefaction-induced ground failures have been a major cause of earthquake-caused damage and destruction to bridges. For example, during the 1964 earthquake at Prince William Sound, Alaska, liquefaction and liquefaction-induced lateral spreading damaged or destroyed 266 bridges (Youd, 1993a; Kachadoorian, 1968; McCulloch and Bonilla, 1970). Other examples of damaged bridges from earthquake-induced ground failures were the Nigata, Japan earthquake in 1964 where four bridges were damaged or destroyed (Youd, 1993a), and the 1991 Limon Province, Costa Rica earthquake where seven bridges collapsed with several others severely damaged (Youd et. al., 1992). The majority of the failed or damaged structures were located at river crossings where the soils tend to be loose, saturated, granular deposits. These soils are highly susceptible to liquefaction and the gently sloping floodplain and incised slopes around these river crossings are vulnerable to liquefaction-induced lateral spread.

Liquefaction by itself generally is not of significant distress to bridges unless the foundation (piles, piers, caissons, etc.) bears on the liquefied layer. Ground failures, including slumps, lateral spread, loss of bearing strength, and excessive settlement, are usually the cause of the damage. Due to the potential for large liquefaction-induced ground movements and the subsequent damage or failure, evaluation of liquefaction and ground failure are an important part of site assessment and structural design of bridges. A screening guide has been developed by Youd (1998) to aid highway engineers by providing a step-by-step guide to the assessment of liquefaction and lateral spread hazards to bridges. The guide assesses liquefaction hazard and then prioritizes bridge sites for further, more detailed investigation. Though of use anywhere, Utah was chosen to test the screening guide because it is seismically active, has a large amount of potentially liquefiable deposits, and major reconstruction of the interstate system is currently underway (began in 1997).

Objective

The objective of this study was to evaluate the liquefaction hazard of bridge sites in Utah and to test and suggest revisions to the screening guide. The principal highway systems, including Interstate and other Federal highways, and some State roads, were screened using the guide. This report describes step-by-step utilization of the screening guide to evaluate liquefaction hazard at bridges along these routes. The bridge sites with significant potential for liquefaction and subsequent ground failure were given a high priority for further investigation. The primary results from this study are a tabulation of bridge sites with high priorities for future investigation and possible remediation. Included are depths and thicknesses of potentially liquefiable layers beneath bridge sites throughout the state of Utah with emphasis on the I-15 corridor, as assessed utilizing the simplified procedure. Calculation of ground displacements other than settlement was beyond the scope of this study so only one example calculation is presented.

The objective of this study was also to conservatively prioritize bridges for future, more detailed investigation. Further investigation is needed to quantify the liquefaction hazard at a site. This study assumed that any site underlain by possibly liquefiable sediment has a high priority for future investigation. It is expected that after more extensive investigation many sites conservatively classed as high priority in this report would actually have low liquefaction hazard.

CHAPTER 2 - REVIEW OF PREVIOUS SCREENING GUIDES

Few formal screening guides have been compiled to screen bridge sites for liquefaction and displacement hazards. Many reports have been written explaining the steps used in the evaluation of soil liquefaction but they lack any mention of bridge site screening. Other reports have been written which outline potential and observed bridge damage by earthquakes with little or no explanation of liquefaction assessment. An effective screening guide should evaluate site factors including soil type, groundwater depth, and seismicity as well as structural factors like foundation and bridge strengths, and ground displacement resistance. Only two guides were found which addressed both of these factors. They are by Tokida and others (1991), and Ferritto and Forrest (1977). The Federal Highway Administration (FHA) (1992) has produced guidelines for bridge design that could be considered a screening guide, but these guidelines were basically taken from the work of Ferritto and Forrest (1977) so the FHA guide was not reviewed. The reports by Tokida and others (1991) and Ferritto and Forrest (1977) are discussed below.

2.1 The Japanese Simplified Procedure (Tokida et. al., 1991)

The Japanese Simplified Procedure was developed to periodically (every three to five years) evaluate liquefaction hazard at existing structures. This procedure is depicted best with the flow chart in figure 1 which shows the principal steps of the liquefaction hazard evaluation for bridges. The four main sections of the screening are the Fundamental Inspection, Site Liquefaction Inspection, Damage Potential Inspection, and the Detailed Stability Inspection.

2.1.1 Fundamental Inspection

The first step in the Japanese simplified procedure is to review the construction period and bridge type. If the bridge is newer than 1971 the structure was designed using a design code that takes into account liquefaction hazard. If the primary structural system is an arch, rigid-frame, or culvert, the bridge is assessed as safe because these types of bridges have not been severely affected by liquefaction in the past. If the bridge does not fall into one of these structural systems or if it is older than 1971, the screening proceeds to the next step as shown in figure 1.

2.1.2 Site Liquefaction Inspection

This section of the Japanese screening guide evaluates the susceptibility of a site to liquefaction. The first step is to collect and analyze borehole data for the bridge site. If the available data indicate that there are no saturated sandy layers in the subsurface, the site is assessed as resistant to liquefaction thus completing the evaluation. However, if there are saturated sandy layers, a factor termed the liquefaction resistance factor, F_L is calculated for each layer. This factor is determined using relationships between the soil's "liquefaction resistance" and the "seismic load on the soil" (Tokida et. at., 1991). These relationships require knowledge of the layers such as mean grain size, total and effective stresses, and density (usually acquired by the standard penetration resistance (STP), test). For sites having layers with F_L less than 1.0,

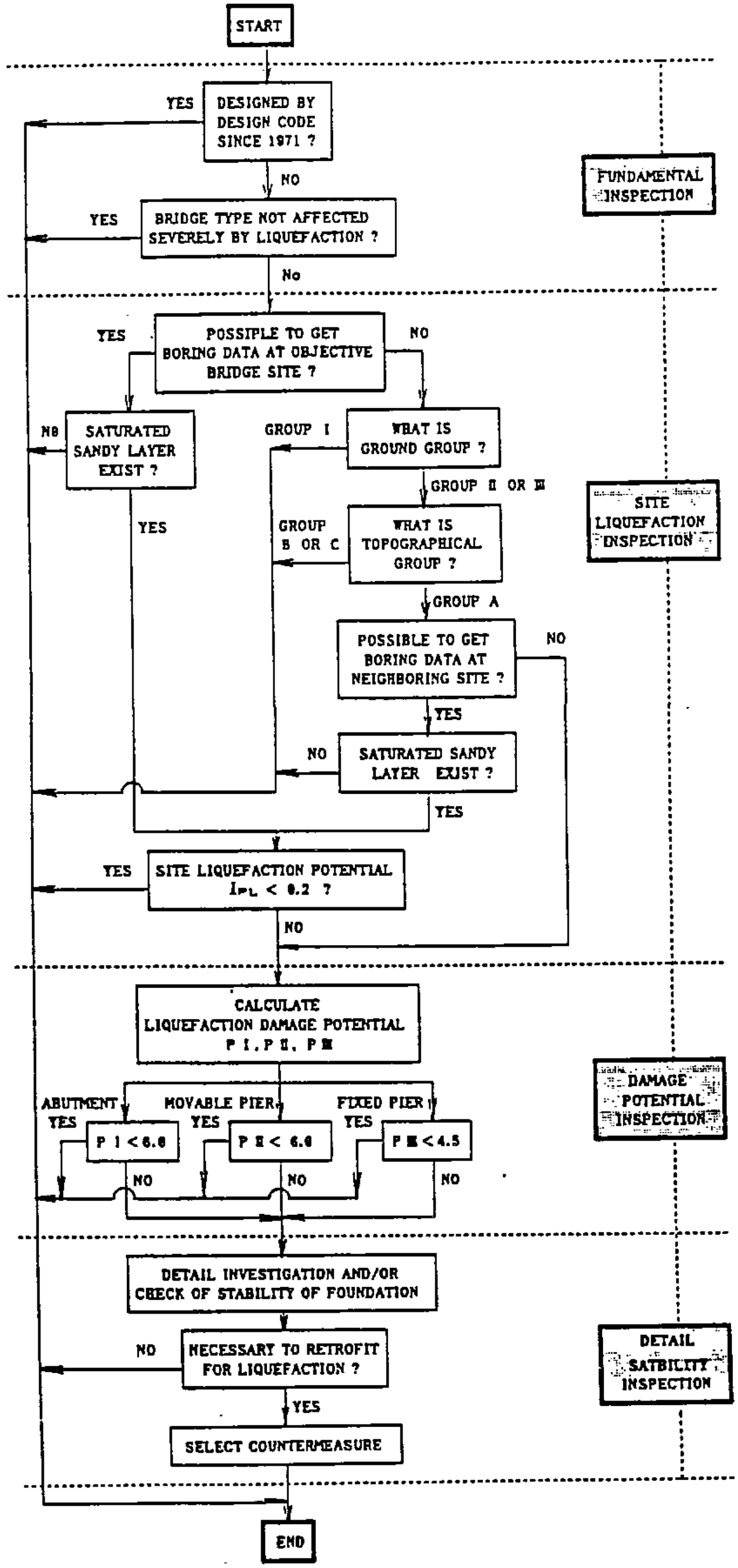


Figure 1: Flow diagram showing steps used to evaluate the earthquake resistance potential of highway bridge foundations (Tokida et. al., 1991)

the liquefaction potential of the entire site, I_{PL}, is calculated. I_{PL} is a weighted average of the thicknesses of the liquefiable layers with more weight given to layers less than 10 m deep. If I_{PL} is less than 0.2, the site is classed as a low liquefaction hazard and the evaluation is concluded. Otherwise, the guide proceeds to the next section of the liquefaction evaluation.

If borehole data are unavailable, the site is evaluated using the general soil type and site topography. Tables 1 and 2 show the Ground Group and Topographical Group classifications. If the soil is of Ground Group I or of Topographical Group B or C the site is assessed as medium to low liquefaction hazard. For Ground Group II or III or Topographical Group A, borehole data from a neighboring site should be examined to determine if it is underlain by saturated sandy layers. If there are sandy layers beneath the nearby site, the liquefaction potential, I_{PL}, is calculated for that site. If the liquefaction potential, I_{PL}, is less than 0.2, both sites are assessed as having a low liquefaction hazard. If I_{PL} is greater than 0.2 the screening evaluation continues to the next step.

Table 1: Ground Group Classification (after Tokida et. al., 1991)

Ground Group		Liquefaction Potential
Group I	Rock, Diluvial Ground (firm)	Low Liquefaction Potential
Group II	Alluvial Ground (medium-stiff)	High Liquefaction Potential
Group III	Soft and Thick Alluvial Ground (soft)	High Liquefaction Potential

2.1.3 Damage Potential Inspection

For all sites with $I_{PL} > 0.2$, a second factor is calculated termed the liquefaction damage potential, PI, PII, or PIII. The Japanese procedure divides the bridges into three types: abutment (PI), movable pier (PII), or fixed pier (PIII). The liquefaction damage potential is a function of the liquefaction potential, I_{PL}, the effects on the structure due to a girder failure, I_C, the girder-support condition on the top of the pier or abutment, I_J, the foundation type, I_F, the bearing condition of the foundation, I_{FS}, the flow of abutment backfill, I_{HB}, and the deformation of surrounding ground, I_{GC}. Table 3 shows the damage potential criteria of the three bridge types. If the potential is less than the value shown in the table, the bridge is classified as not vulnerable to damage due to liquefaction, otherwise the site is investigated for stability.

2.1.4 Detailed Stability Inspection

This final step in the Japanese screening guide states that a detailed inspection should be conducted to "calculate the stability and resistance of foundations" to the effects of liquefaction. It does not, however, suggest any method for doing this. If the bridge is found to be stable under liquefied conditions, the site is classified as having low hazard. If the bridge is unstable or weak, a retrofit is necessary and countermeasures against liquefaction or liquefaction-induced ground failures should be selected and implemented.

Table 2: Topographical Group Classification (Tokida et. al., 1991) [sic]

Groups		Topographical Condition
A	High Liquefaction Potential	Reclaimed/Filled-up land Banked-up ground on the alluvial low ground Banked-up ground on the former water/sea area Slight heights on the former river bed Land by drainage Tidal Flat Coastal plain Delta Natural levee Former river bed (Former) marsh (Former) moat Flood plain Sand dune Sand bar Back low land
B	Medium Liquefaction Potential	Except for Group A and Group C (Banked-up ground except for A, Fan etc.)
C	Low Liquefaction Potential	Terrace Plateau Heights Hill Slope Mountain

Table 3: Damage potential of bridges due to liquefaction (Tokida et. al., 1991)

Foundation Type	Index Range	Explanation
Abutment	PI < 6.0	Damage induced by liquefaction can be estimated to be small
	PI ≥ 6.0	Be in danger of severe damage. Detail investigation should be necessary (sic)
Moveable Pier	PII < 6.0	Damage induced by liquefaction can be estimated to be small.
	PII ≥ 6.0	Be in danger of severe damage. Detail investigation should be necessary. (sic)
Fixed Pier	PIII < 4.5	Damage induced by liquefaction can be estimated to be small.
	PIII ≥ 4.5	Be in danger of severe damage. Detail investigation should be necessary. (sic)

2.1.5 Limitations of the Japanese Simplified Procedure (Tokida et. al., 1991)

The Japanese simplified screening procedure (Tokida et. al., 1991) has some limitations which prevent its use here in the U.S. The first limitation is that the design earthquake is never taken into account anywhere in the procedure, i.e. magnitude and acceleration are never mentioned. The likely reason for this omission is the general uniformity of the high earthquake hazard in Japan. In the U.S., a bridge that is located a large distance from a fault or other seismic source zone may never experience earthquake shaking strong enough to cause liquefaction. However, by strict interpretation of the Japanese criteria, such bridges would still be considered hazardous if the soil beneath it is susceptible to liquefaction.

The second limitation is the vagueness of many of the indices. The Ground Group Classification and the Topographical Group Classification indices need better correlations between liquefaction hazard and soil types, groundwater conditions, and topographic settings. For example, the criteria for Ground Group III states that if the ground consists of soft and thick alluvial deposits, the liquefaction potential is high. However, clays and plastic silts also meet these criteria but have low susceptibility to liquefaction. The indices need to be more specific to be more widely applicable.

The third limitation with the Japanese method is the vague criteria for determining the liquefaction-induced damage potential. This factor is determined from structural aspects of the bridge and from vague indices such as the Ground Group. The structural characteristics of the bridge like the "effect of device against fall of a girder...effect of girder-support condition on the top of pier/abutment," or "effect of foundation type" are all assigned a factor from one to six and multiplied by the site liquefaction potential to give a damage potential. Ground failure modes like settlement, lateral spread, or loss of bearing capacity are not mentioned or calculated in the guide. Thus, if the structure has certain structural characteristics, the damage potential will be high regardless of the type or amount of liquefaction-induced ground failure. Also, because the guide does not require the calculation of ground displacements, the usefulness of the screening guide is limited for the design of remedial measures.

The last limitation of the Japanese screening method is the lack of guidance regarding sites with no borehole data. Tokida and others (1991) state that if borehole data are unavailable, a neighboring site should be analyzed. If neighboring borehole data are unavailable, however, they do not outline what should be done to assess the site. Their screening guide needs a step which says that if no borehole data can be found, the site should be assessed as having a liquefaction potential and should be further evaluated.

In summary, the Japanese Procedure by Tokida and others (1991) fails to account for variations in earthquake loading, and is vague in many of the "indices" required for calculation. It also fails to quantify types or amounts of ground displacement or potential damage at a site, and to specify limits of displacement a bridge might withstand without damage. The procedure also fails to outline additional steps or suggestions for bridge sites with insufficient data.

2.2 Liquefaction Potential by Ferritto and Forrest (1977)

The procedure developed by Ferritto and Forest (1977) was adopted by the Federal Highway Administration in its *Foundation and Abutment Design Requirements* (1992). The Ferritto and Forrest report consists of two volumes: the first volume discusses the theoretical concepts, and the second is a planning guide. The following is a review of the planning guide only.

According to the Planning Guide, there are several variables that must be determined to evaluate the liquefaction hazard at a bridge site. These variables are as follows:

1. Site Earthquake
2. Site Definition
3. Liquefaction Evaluation
4. Consequences of Liquefaction
5. Risk Assessment and Damage Minimization

2.2.1 Site Earthquake

According to Ferritto and Forrest (1977) the first step in bridge liquefaction evaluation is to determine the earthquake-induced loading for the site. This loading is determined from the magnitude and acceleration of the design earthquake. They recommend the design magnitude be determined using probability curves of the form:

$$P(M)=e^{-U(M)t} \tag{1}$$

where:

U(M) = number of events per year greater than or equal to magnitude M
t = life of the structure
P = probability of an earthquake equal to or exceeding M in time t

An example of these curves is shown in figure 2. Ferritto and Forrest (1997) explain that when selecting the design earthquake magnitude, "consideration must be given to all faults in the area since a large event on a distant fault may produce lower levels of site acceleration than a closer smaller event; however, the number of cycles produced (or the duration of strong shaking) may be greater. Remember the combination of both stress level and number of cycles (or duration) affect liquefaction potential."

Ferritto and Forrest (1977) suggest the site acceleration be determined from relationships between both the distance from the fault and earthquake magnitude. They discuss methods by Schnabel and Seed (1972), Trifunac and Brady (1975), and Algermissen and Perkins (1976) for determining the acceleration at a site. Once the magnitude and acceleration have been calculated, the site must be investigated and defined.

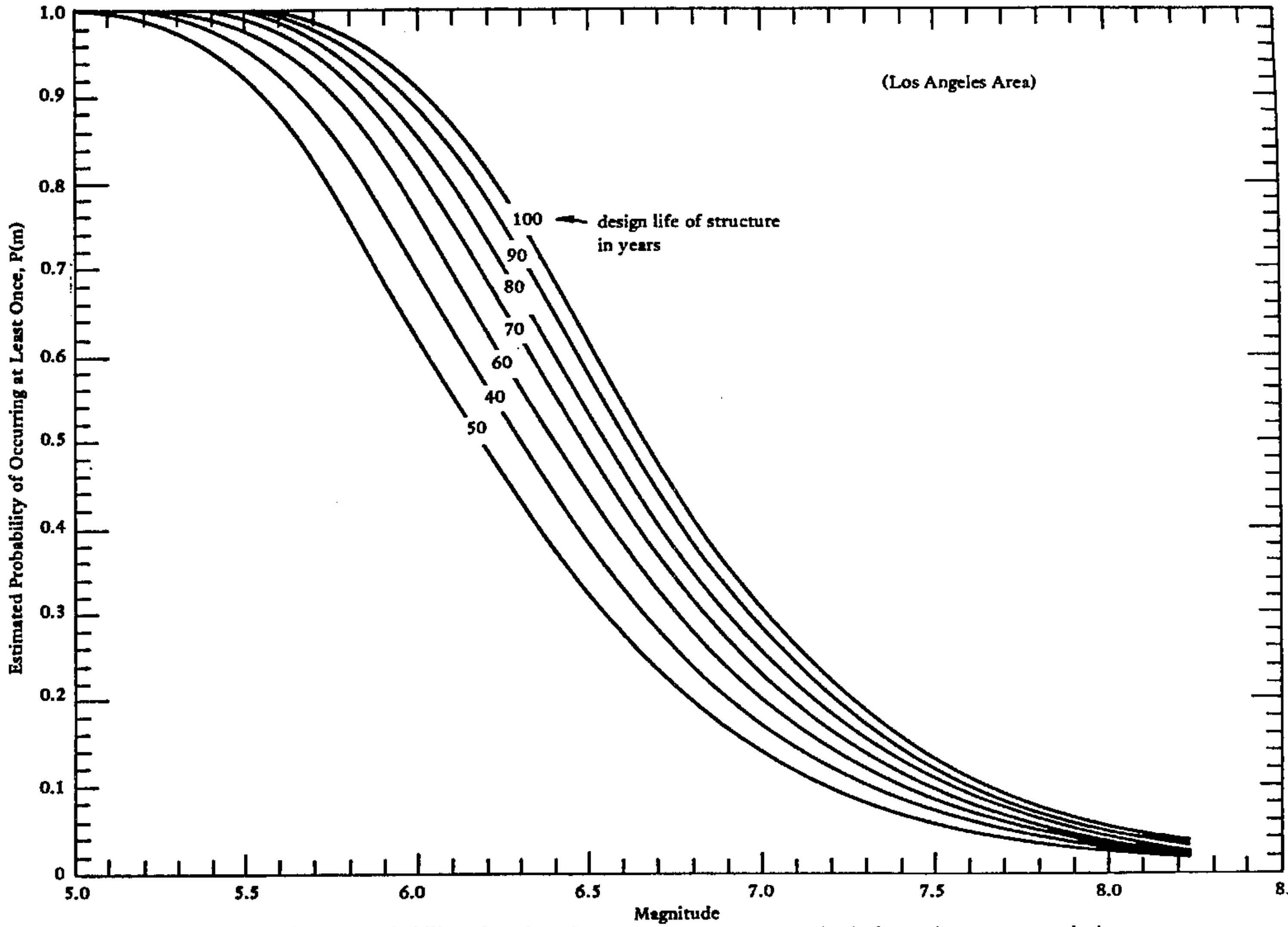


Figure 2: Estimated probability of earthquake occurrence versus magnitude for various structure design lives (Ferritto and Forrest, 1977)

2.2.2 Site Definition

A detailed site investigation is needed to provide information about the subsurface conditions such as soil types, densities, strengths, and the groundwater table. This information can be gathered by taking samples using shelby tubes, the standard penetration test (SPT), or the cone penetration test (CPT). The samples should be tested in the laboratory to properly classify the soil and its characteristics. These tests should determine the Atterberg limits, grain size distributions, and visual classifications (Ferritto and Forrest, 1977).

2.2.3 Liquefaction Evaluation

Ferritto and Forrest (1977) propose two methods for determining the liquefaction potential of a saturated sandy layer. The first method uses field data compiled during previous earthquakes. This data is used to create a chart that shows the SPT resistance for areas that did or did not liquefy during an earthquake. The chart is then used to delineate conditions where liquefaction may or may not occur in the future. Empirical relationships tend to be inaccurate, therefore, Ferritto and Forrest recommend the second method.

The second approach for liquefaction evaluation determines a factor of safety by dividing the stress conditions required to cause liquefaction at a site with the stress conditions created by the earthquake. This method is termed the "simplified procedure" and was developed by Seed and Idriss (1971) and updated by Seed and others in 1985. Section 4.3 of this report outlines this procedure in detail. Another method used to determine the liquefaction potential of soils was developed by Seed and Peacock (1971). This procedure analyzes the liquefaction hazard using a relationship between the relative density and ground acceleration. A summary of this procedure is shown in table 4. Once the potential for liquefaction has been established, the next step is to determine its effects.

Table 4: Correlations between ground accelerations, relative density, and liquefaction (Seed and Peacock, 1971).

Maximum Ground Surface Acceleration	Liquefaction Very Likely	Liquefaction Potential Depends on Soil Type and Earthquake Magnitude	Liquefaction Very Unlikely
0.10 g	Dr < 33	33 < Dr < 54	Dr > 54
0.15 g	Dr < 48	48 < Dr < 73	Dr > 73
0.20 g	Dr < 60	60 < Dr < 85	Dr > 85
0.25 g	Dr < 70	70 < Dr < 92	Dr > 92

2.2.4 Consequences of Liquefaction

This section of Ferritto and Forrest's report (1977) gives equations for calculating displacement for unlimited flow conditions (free flow velocity times the elapsed time of ground shaking) and calculation of foundation capacity reduction due to liquefaction. It also mentions (but does not explain in detail) other effects such as liquefaction with limited displacement, bearing capacity failures, subsidence, and bridge responses to liquefaction. They suggest the use of models and finite element analyses to evaluate the bridge's response to liquefaction.

2.2.5 Risk Assessment and Damage Minimization

The risk to a structure should be determined based on the probability of liquefaction and the possible damages to the bridge. If the risk is significant, Ferritto and Forrest list the following criteria that will minimize the bridge damage:

1. Site selection: Using topographical, geological, and geotechnical factors as a basis, avoid building a bridge on sites which may be susceptible to liquefaction. These are sites with shallow water tables and thick layers of cohesionless materials.

2. Site improvement: The liquefaction potential of a site can be reduced by lowering the groundwater level, increasing the relative density of the soil, or increasing the permeability of the site.

3. Bridge design: Design structures to reduce the level of working stresses. Strengthen the design to withstand larger ground movements. Strengthen the foundation with deeper footings and more piles.

2.2.6 Limitations of the Ferritto and Forrest Method (1977)

The screening procedure developed by Ferritto and Forrest (1977) proves to be useful for both existing and future bridge evaluation but does have some limitations. The first limitation of their method is the lack of non-site-specific screens such as geologic and liquefaction maps, groundwater table depths, etc. Many sites can be classified as having a low liquefaction hazard based on this data alone. Another shortcoming of their work comes from the age of the equations and basis for their procedure. The current methods for liquefaction evaluation have changed in 20 years since Ferritto and Forrest (1977) did their research.

The basic outline of their screening guide is effective because it evaluates site factors such as soil type, groundwater elevation, and seismic characteristics, as well as bridge factors such as foundation and bridge strengths, and ground displacement resistance. However, the guide as a whole is obsolete because it is based on out-of-date equations and procedures. Their liquefaction calculations are based on a version of the "simplified procedure" that is older than that explained in section 4.3. The Federal Highway Administration's screening guide is derived from Ferritto and Forrest's work so even though it is much more recent (1992 as opposed to 1977), it is also outmoded.

APPLICATION OF THE SCREENING GUIDE

The screening guide developed by Youd (1998) provides a systematic application of standard criteria for assessing liquefaction hazard and for prioritizing sites for further investigation. The guide proceeds from simple low-cost evaluations, requiring little site-specific data, to more complex, time consuming, and rigorous analyses. The screening guide requires application of existing information, such as past liquefaction hazard analyses, geologic maps, foundation investigation reports, and does not require development of new site data. By application of the simpler evaluations first, bridge sites in low hazard areas may be classified as low hazard with minimal time and effort. Only bridges in the more vulnerable settings need to be analyzed with more time consuming site-specific evaluations.

At each step in the analysis, a conservative assessment of hazard is made. Where there is clear evidence that liquefaction or damaging ground displacements are very unlikely, the site is classified as low liquefaction hazard and low priority for further investigation (Priority IV). At that point the evaluation is complete for that bridge site. If there is evidence that a liquefaction hazard may exist, the site is classed as possibly liquefiable and the analysis proceeds to the next step. The evaluation proceeds step by step until the bridge site is classified as either hazardous, nonhazardous, or insufficient available information to evaluate the hazard. If available information is insufficient, geologic, hydrologic, topographic, and degree of importance information are used to prioritize the bridge site for further investigation. The final outcome of the screening is an assignment of each bridge site to one of the following four categories or priorities for further investigation and possible mitigation:

<u>Priority I sites</u>: Bridge sites assigned the highest priority for further investigation and possible mitigation are those likely to be underlain by liquefiable sediment that is capable of inducing damaging ground or foundation displacement. Liquefiable sediment was either confirmed or the information was insufficient to eliminate the possibility of liquefiable sediment beneath these sites. These sites are at localities of likely ground failure, including crossings over rivers or other bodies of water, near steep slopes, or approached by high (greater than 5 m) embankments. These sites are classed as potentially hazardous because of the high incidences of bridge damage in these settings during past earthquakes.

<u>Priority II sites</u>: Bridge sites with the second highest priority for further investigation and possible mitigation are localities confirmed to be underlain by liquefiable sediments or sensitive clay, but where the available site information is insufficient to fully evaluate ground failure or foundation instability hazards. A second criterion for Priority II sites is that they are located away from rivers, other bodies of water, or steep slopes (otherwise they would be classed as Priority I sites). Liquefaction at these sites could cause ground settlement and possibly foundation instability, but damaging lateral ground displacements are unlikely. These sites are given moderately high priority for further investigation.

<u>Priority III sites</u>: Bridge sites with third priority for further investigation are those with insufficient compiled geotechnical information to fully assess the liquefaction hazard. These sites are also restricted to generally flat terrain that is away from rivers or other bodies of water,

steep slopes or high approach embankments. Further investigation is required to assess ground settlement and foundation instability hazards, but lateral ground displacements are not likely to develop. Priority for further investigation of these sites is moderate to low.

Priority IV: Bridge sites with the lowest priority for further investigation are those where the screening evaluation indicated very low liquefaction or ground failure hazard. These bridges are categorized as very low priority for further investigation or mitigation.

The general screening procedure, as outlined in the flow chart reproduced in figure 3, consists of three sequential levels of evaluation--regional screening, site-specific evaluation, and assessment of ground failure and foundation instability hazard. Each succeeding level of investigation requires more detailed information and more complex and more time consuming analyses. Each level contains several intermediate steps with a decision required at the end of each step. If the finding is that liquefaction or ground failure potential is very low, the site is classed as low liquefaction hazard and low priority for further investigation (Priority IV site), and the evaluation is complete for that site. If the conclusion is that a possible hazard exists, the evaluation continues to the next step.

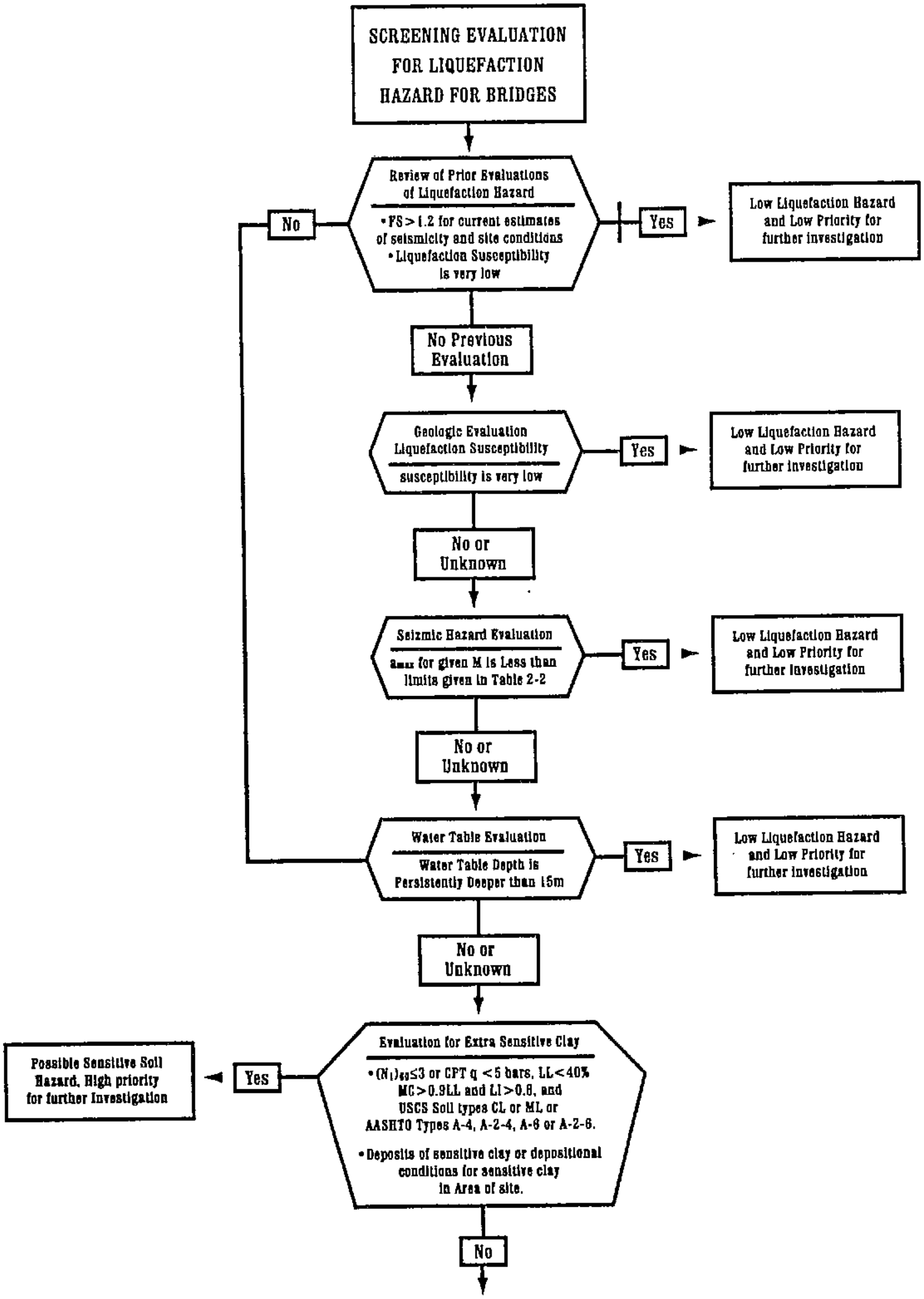


Figure 3: Flow diagram showing steps and criteria for screening of liquefaction hazard for highway bridges (after Youd, 1998).

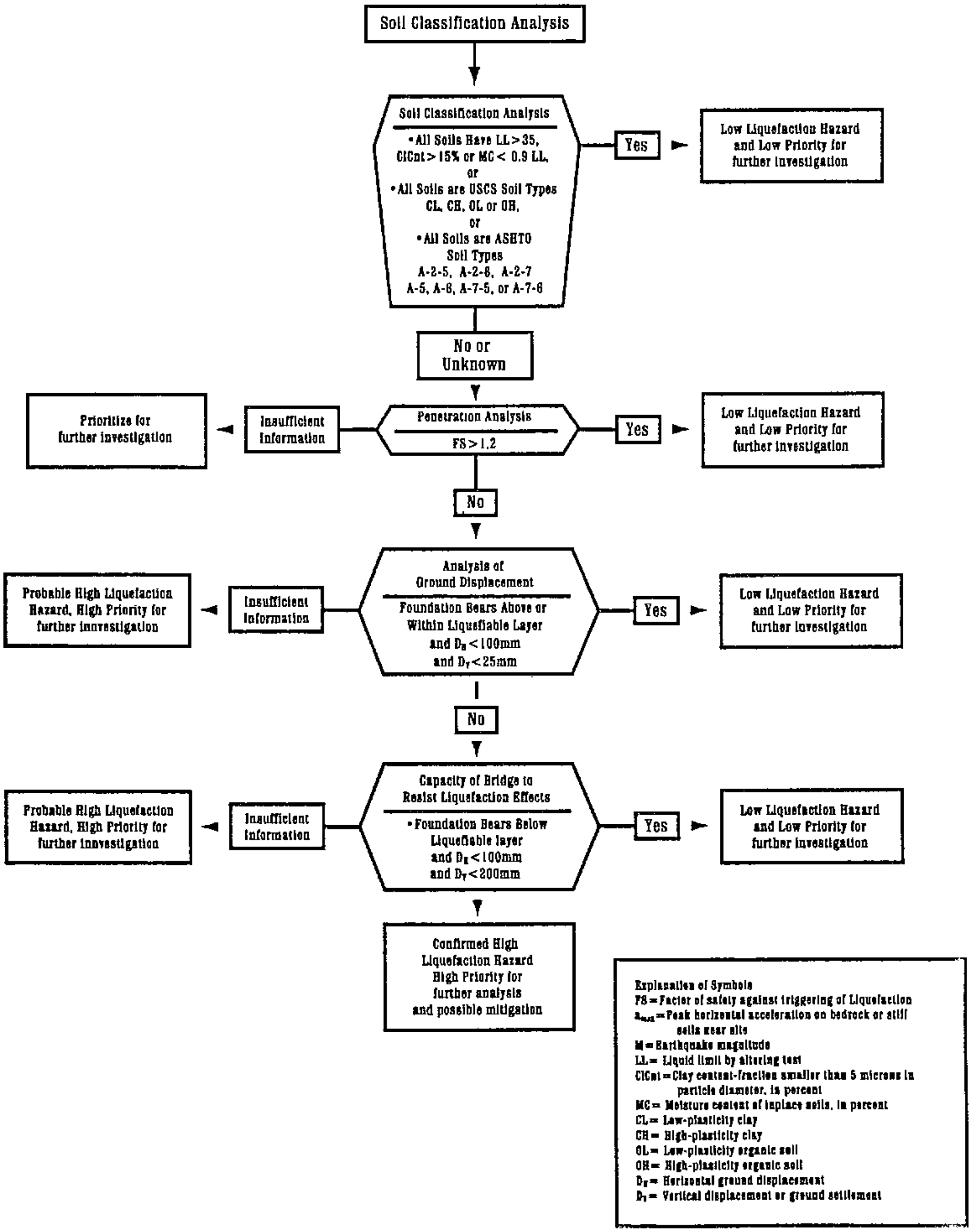


Figure 3 (continued): Flow diagram showing steps and criteria for screening of liquefaction hazard for highway bridges (after Youd, 1998).

CHAPTER 3 - REGIONAL SCREENING

Regional screening involves assessment of liquefaction potential from the general setting of the bridge site rather than from site-specific information such as blow counts, Atterberg limits, etc. The first four steps of the flow chart in figure 3 review prior liquefaction evaluations, geologic conditions, seismic characteristics, and water table elevations. In many instances, a bridge location may be shown to have low liquefaction potential based on regional information alone.

3.1 Prior Liquefaction Evaluation

Youd (1998) explains that the first step in using the screening guide is to evaluate any available information on liquefaction susceptibility at or near the bridge site of concern. This evaluation includes inspecting prior liquefaction cases at or near the site (liquefaction may occur repeatedly at a site) as well as examining any previous liquefaction studies performed for the area in question. These studies should be checked to be sure they are conservative and that the screening criteria used is similar to those used in the screening guide. If bridge sites lie in areas classified by prior evaluations as having "low" liquefaction potential, they are considered as having a low liquefaction potential and no significant damage would be expected during an earthquake. If the bridge site being screened is located in areas zoned as "high or very high" then the screening should pass directly to the site-specific stage of the analysis (see chapter 4). For bridge sites that are listed in these studies as "moderate" liquefaction potential or sites that are not located in areas previously studied, the screening guide should be followed to the next step as shown in figure 3.

This first step of the screening guide was applied to Utah by first looking at recorded cases where liquefaction has taken place. Review of available historical information shows that only a few cases of liquefaction have been recorded in Utah. Surface effects of liquefaction were reported at several sites following the 1937 Hansel Valley, the 1962 Cache Valley, and the 1992 Washington County earthquakes. The liquefaction occurrences in 1937 and 1962 were isolated and not very close to any interstate freeways, but the earthquake in 1992 caused liquefaction within a few kilometers of I-15. Nearly 0.3 km from I-15 over the Virgin River, lateral spread displacements between 94 mm and 165 mm were observed. A lateral spread displacement of 493 mm was measured along the Virgin River bank about 2 km upstream of the I-15 bridge (Black et. al., 1992). These lateral spread displacements along with numerous sand boils along the river demonstrate the need for a more detailed evaluation of the bridge sites in the area.

The next step was to look at any prior studies of liquefaction potential in Utah. Mabey and Youd (1989) compiled probabilistic liquefaction severity index (LSI) maps for the state. The LSI maps provide estimates of maximum lateral spread displacements that are likely to occur at sites underlain by liquefiable sediments. These maps were useful to identify potential liquefaction hazards in large sections of the state. Other studies that were found include an extensive set of liquefaction hazard maps for most of the seismically active Wasatch Front (Anderson et. al., 1994a,b,c,d,e). Maps were available for Box Elder, Weber, Cache, Davis, Salt

Lake, and Utah Counties as well as sections of many of the alluvial valleys south of Utah County such as Juab Valley, Sanpete Valley, Sevier Valley and others. All of these studies review the potential for liquefaction and delineate areas of potential hazard.

Example Screening of Utah County Using Prior Liquefaction Evaluations

Implementation of the first step of the screening guide was illustrated for Utah County. No historical accounts of liquefaction have been reported for Utah County so a review was made of prior studies of liquefaction. Mabey and Youd (1989) show that the lower flat lands of Utah County (the part where Interstate 15 is located) has a high liquefaction hazard based on the LSI. The liquefaction potential maps by Anderson and others (1994c), shown in figure 4, confirm this hazard. Starting at the north end of the county, I-15 passes over an upland ridge, called Point of the Mountain, an area zoned as very low to low liquefaction potential. The six bridges located in this zone, between the Salt Lake County line and Lehi, are therefore classified as having a low liquefaction hazard and low priority for future study. East of Lehi the interstate passes through a narrow zone of moderate hazard and into a broader zone of high hazard. The freeway continues south in this high zone until it reaches State Road 146 (Geneva Rd.) where it again passes through a narrow moderate zone into a zone of low hazard. Eight bridges lie in the high hazard zone and three bridges lie in the moderate zone. All 11 bridges have sufficient probable hazard to require site-specific evaluations (see sections 4.1-4.4 of this report).

The zone of low liquefaction potential along the I-15 near 800 North in Orem contains two bridges that are classed as low hazard and low priority for further study. Following the interstate south, it crosses a moderate zone until it reaches the southern end of Provo. Nine structures lie in this moderate zone. These bridge sites require additional investigation using procedures outlined in sections 4.1-4.4 of this report. South of Provo, the highway route passes through an extensive area, from Provo to Hwy. 6 (near Payson), characterized by high liquefaction potential. The 18 bridges in this zone were analyzed using the site-specific methods contained in chapter 4. At Highway 6, the freeway enters into a zone characterized as moderate to moderate-low potential. Five bridges are located in this zone which were evaluated further in the subsequent steps of the screening guide. South of Highway 6, near Payson, I-15 enters a zone classified as very low potential. This zone continues into and through most of Juab County (Anderson et. al., 1994c; Anderson et. al., 1994e). This segment of Utah County contains only three bridges. All three of these structures are classified as low hazard and low priority for further investigation. In summary, there are 54 bridges along I-15 in Utah County. Using prior liquefaction analyses by Mabey and Youd (1989) and Anderson and others (1994c), 11 of these sites were identified as having little or no liquefaction hazard, 17 sites were classed as moderate hazard, and 26 classed as high to very high liquefaction hazard. The latter 43 bridge sites were further analyzed using site-specific procedures, as indicated on the flow chart in figure 3.

3.2 Geologic Analysis

For areas where previous liquefaction evaluations do not exist, a geologic analysis may be used to identify non-hazardous sites. This step of the screening guide evaluates liquefaction

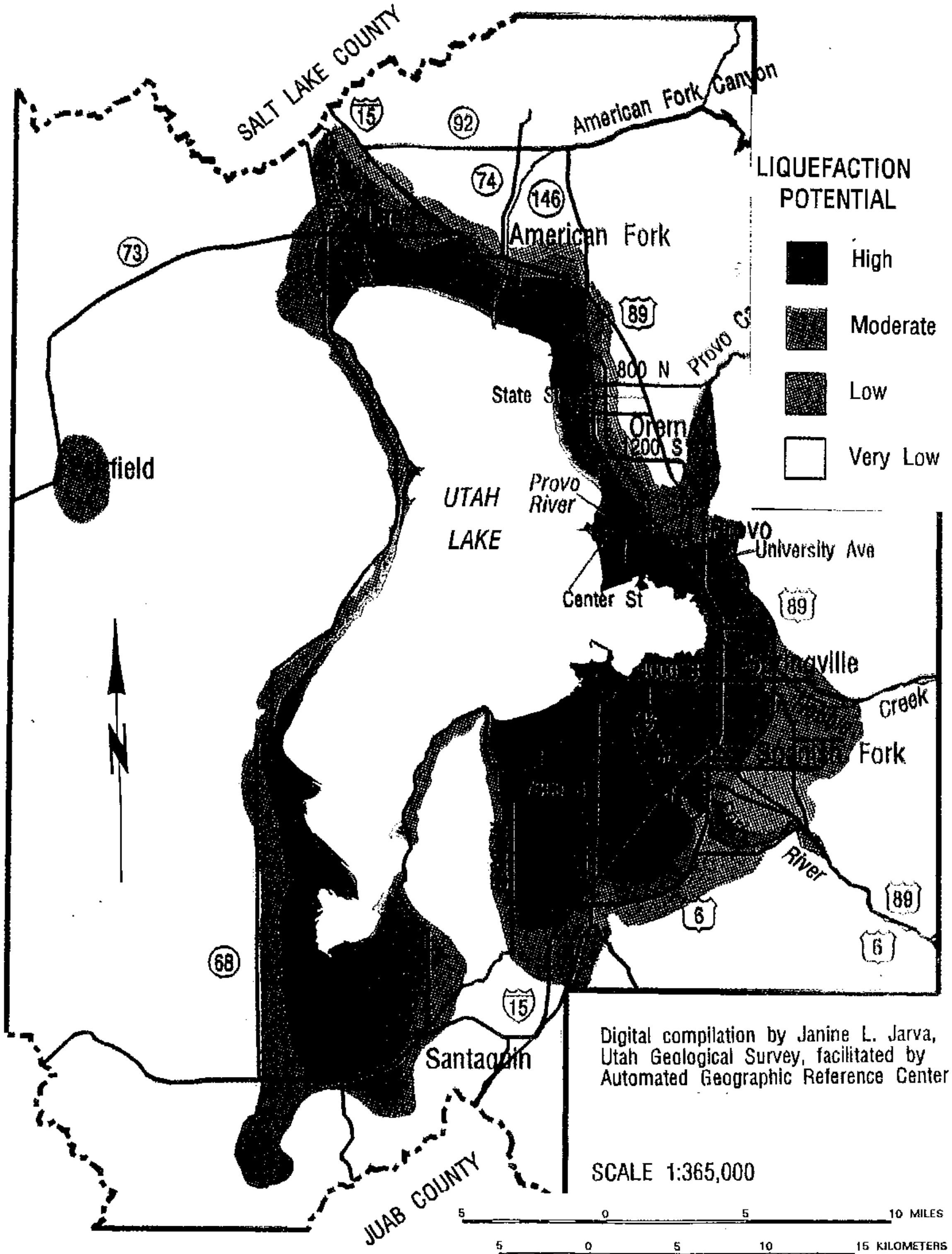


Figure 4: Liquefaction potential map for a part of Utah County, Utah (after Anderson et. al., 1994c)

susceptibility based on geologic units and geologic age. Liquefaction occurs in a narrow range of sedimentary environments. Youd (1998) explains that to liquefy, soils must be uncemented, granular, saturated, and relatively recently deposited (Holocene and late Pleistocene deposits). Detailed geologic maps have been compiled for many areas which adequately define the sediments beneath bridge sites for liquefaction hazard evaluation. Maps that lump together all unconsolidated sediment as Quaternary alluvium (Qal) are generally not useful because they do not adequately distinguish sediment types and ages (i.e. loess, glacial till, flood plain alluvium, etc.). To evaluate liquefaction susceptibility, geologic units on the maps should be compared with those listed in table 5. This table correlates types and ages of sediments with their general susceptibility to liquefaction.

Where Quaternary geologic maps are unavailable, a local map may be created from a review of geologic literature, analysis of soil logs contained in foundation investigation reports, air photo interpretation, or from site reconnaissance visits. If the geologic map indicates potentially liquefiable sediments or if inadequate geologic information is available, the investigation should proceed to the next step of the screening guide. If the geologic maps or foundation reports indicate nonliquefiable soils, the site may be screened as low hazard and given low priority for further study (Youd, 1998).

Example of Geologic Evaluation for Tooele County

Liquefaction potential maps have not been compiled for Tooele County; thus, available geologic maps and reports were utilized. Many parts of the county are underlain by nonliquefiable, pre-Pleistocene rocks and sediment. Currey and others (1984), however, show that the I-80 route lies almost entirely on Lake Bonneville sediment of late Pleistocene and Holocene age. From table 5, the age classification of these sediments show them to be potentially liquefiable. The table shows that Holocene lacustrine and playa deposits have a moderate susceptibility to liquefaction. Because these sediments could possibly liquefy, based on geologic criteria, they were further evaluated with site-specific screening procedures.

3.3 Seismic Evaluation

The next step in the regional screening, as shown in figure 3, is seismic evaluation. This step examines the location of a bridge relative to seismic source zones to determine whether the intensity and duration of possible strong ground shaking is sufficient to induce liquefaction. If the ground shaking potential is not sufficient to induce a large increase in pore water pressures, the soil will not liquefy, even if it is loose and saturated. Two seismic factors, the earthquake magnitude (M) and the peak horizontal acceleration at the ground surface (a_{max}), are needed to calculate liquefaction potential at a site. As an initial screen, Youd (1998) suggests that conservative values of magnitude and maximum acceleration for the region where the site is located should be compared with the values listed in table 6. If a_{max} is below the value listed for a given magnitude, the area can be classed as low liquefaction hazard and low priority for further investigation. For those areas that are higher than the listed values, the screening should proceed to the next step.

Table 5: Estimated susceptibility of sedimentary deposits to liquefaction during strong seismic shaking (after Youd and Perkins, 1978).

Type of deposit	General distribution of cohesionless sediments in deposits	Likelihood that Cohesionless Sediments, When Saturated, Would be Susceptible to Liquefaction (by Age of Deposit)			
		< 500 yr.	Holocene	Pleistocene	Pre-Pleistocene
(1)	(2)	(3)	(4)	(5)	(6)
(a) Continental Deposits					
River channel	Locally variable	Very High	High	Low	Very Low
Flood plain	Locally variable	High	Moderate	Low	Very Low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very Low
Marine terraces and plains	Widespread	-	Low	Very Low	Very Low
Delta and fan-delta	Widespread	High	Moderate	Low	Very Low
Lacustrine and playa	Variable	High	Moderate	Low	Very Low
Colluvium	Variable	High	Moderate	Low	Very Low
Talus	Widespread	Low	Low	Very Low	Very Low
Dunes	Widespread	High	Moderate	Low	Very Low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very Low	Very Low
Tuff	Rare	Low	Low	Very Low	Very Low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very Low	Very Low
Sebka	Locally variable	High	Moderate	Low	Very Low
(b) Coastal Zone					
Delta	Widespread	Very High	High	Low	Very Low
Estuarine	Locally variable	High	Moderate	Low	Very Low
Beach					
High wave energy	Widespread	Moderate	Low	Very Low	Very Low
Low wave energy	Widespread	High	Moderate	Low	Very Low
Lagoonal	Locally variable	High	Moderate	Low	Very Low
Fore shore	Locally variable	High	Moderate	Low	Very Low
(c) Artificial					
Uncompacted fill	Variable	Very High	-	-	-
Compacted fill	Variable	Low	-	-	-

The screening guide describes both a deterministic and a probabilistic method for determining M and a_{max} (Youd, 1998). The deterministic analysis uses empirical relationships between fault length, fault slip etc. to estimate M (Wells and Coppersmith, 1994). For rock or stiff to moderately stiff sites, a_{max} is estimated from correlations between peak ground motion parameters and the distance from the seismic source (Boore et. al., 1993).

The probabilistic analysis utilizes regional maps of expected peak acceleration and expected earthquake magnitudes. This procedure is particularly useful for areas in which specific seismic sources, such as active faults, have not or cannot be easily identified. To be conservative, the probabilistic analysis should use values of a_{max} with 10 percent or less probability of exceedence in 250 years. To be consistent with a_{max}, the chosen earthquake magnitude should have a recurrence interval less than 0.0004 per 1000 km^2/yr (once in 2,500 years). Figure 5 presents a map modifying the data by Hanson and Perkins (1995) showing magnitudes with 0.004 probability per 1,000 km^2 of occurrence per year. The U.S. Geological Survey has published maps which show contours of a_{max} with 10 percent probability of exceedence in 250 years (Algermissen et. al., 1990). These maps are continually being revised and updated so the most recent map should be used.

Table 6: Minimum earthquake magnitudes and peak horizontal ground accelerations, with allowance for local site amplification, that are capable of generating liquefaction in very susceptible natural deposits (after Youd, 1998).

Earthquake Magnitude Mw	Liquefaction Hazard for Bridge Sites	
	Soil Profile Types I and II (stiff sites)	Soil Profile Type III (soft sites)
$M < 5.2$	Very low hazard for $a_{max} < 0.4g$	Very low hazard for $a_{max} < 0.1g$
$5.2 < M < 6.4$	Very low hazard for $a_{max} < 0.1g$	Very low hazard for $a_{max} < 0.05g$
$6.4 < M < 7.6$	Very low hazard for $a_{max} < 0.05g$	Very low hazard for $a_{max} < 0.025g$
$M > 7.6$	Very low hazard for $a_{max} < 0.025g$	Very low hazard for $a_{max} < 0.025g$

Example of Use of Seismic Factors to Screen Sites

Much of Utah is located in the Intermountain Seismic Belt as shown in figure 6. The Intermountain Seismic Belt contains several major faults and is characterized by relatively high seismicity. Major transportation routes and population centers lie within a few kilometers of many of the major faults. The longer faults are capable of producing earthquakes with magnitudes between the range of 7.0 and 7.5. To determine magnitudes and peak accelerations for this study, the probabilistic method was used. The map of Algermissen and others (1990) shown in figure 7 was used to estimate the peak acceleration and a modification of the map by Hanson and Perkins (1995) (figure 5) was used to estimate the magnitude. Some areas of the State were quickly screened as low hazard because combinations of M and a_{max} are lower than the

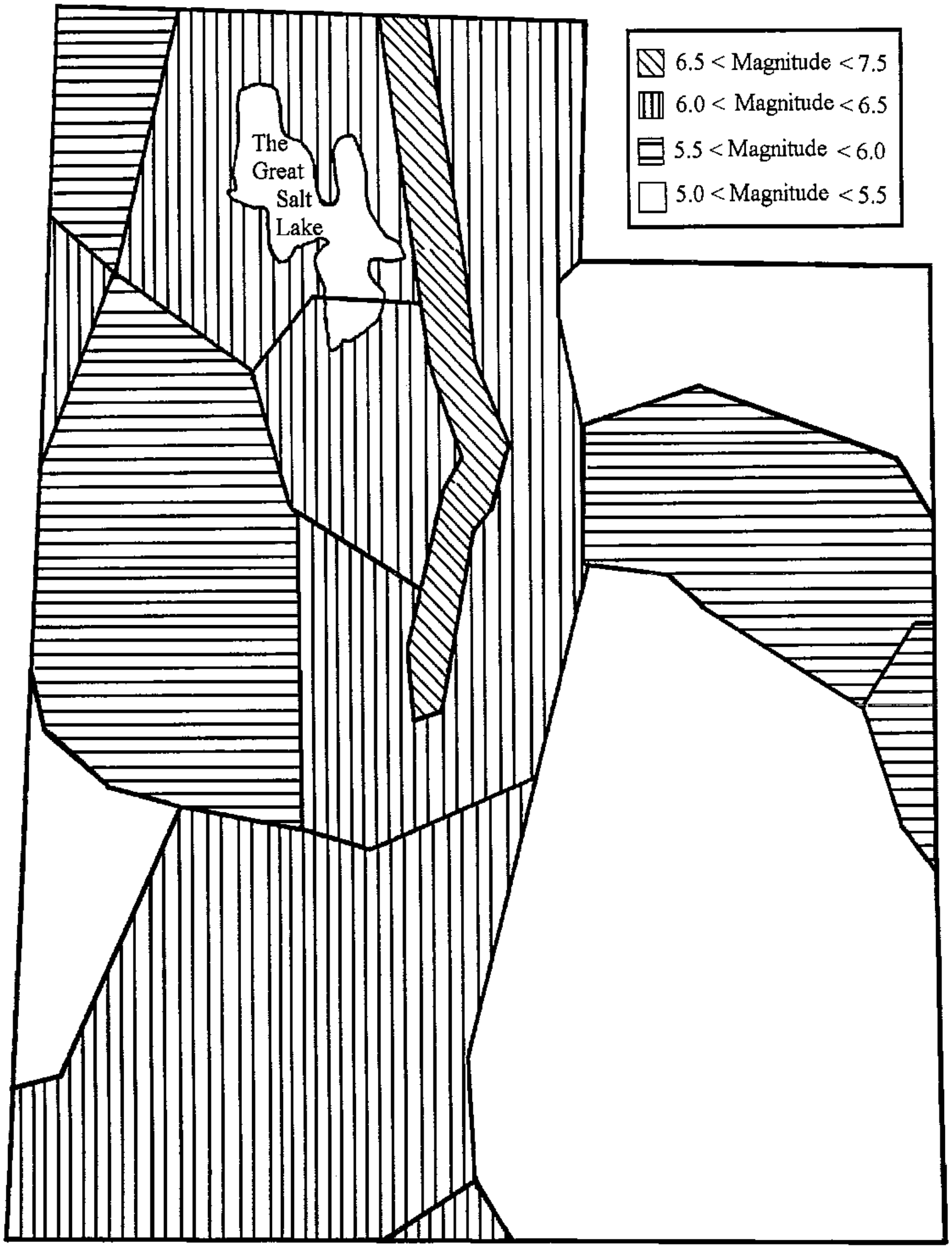


Figure 5: Map of Utah showing seismic source zones and maximum earthquake magnitudes with <0.0004 annual rate of occurence per 1,000 km^2. (>2,500-year reccurence interval) (Data from Hanson and Perkins, 1995)

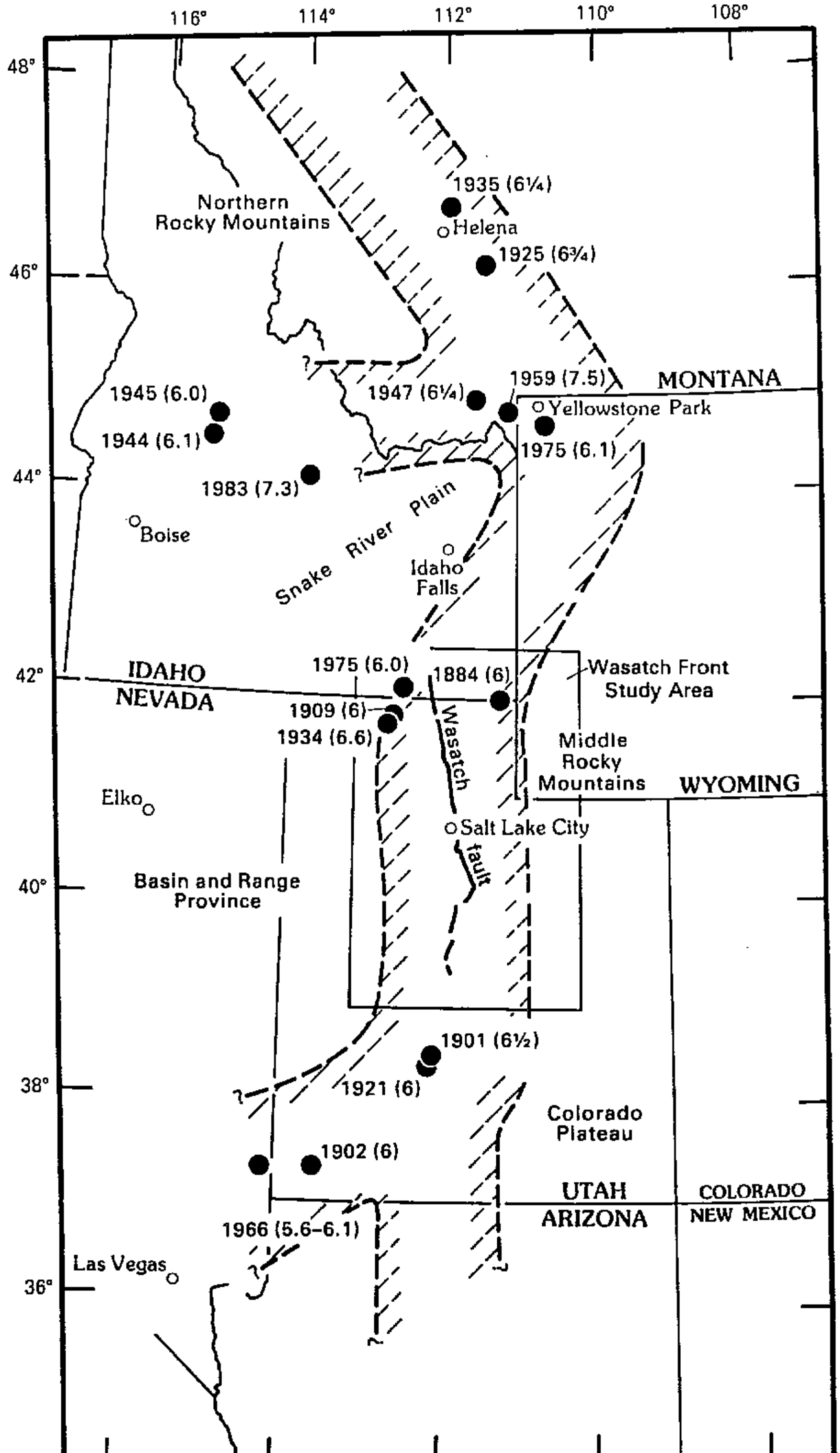


Figure 6: Setting of the Wasatch Front with respect to the Intermountain Seismic Belt (hachured zone) and the epicenters of historical earthquakes of magnitude 6.0 and greater (solid circles). Year and magnitude are labeled for each earthquake. (after Arabasz et. al., 1992)

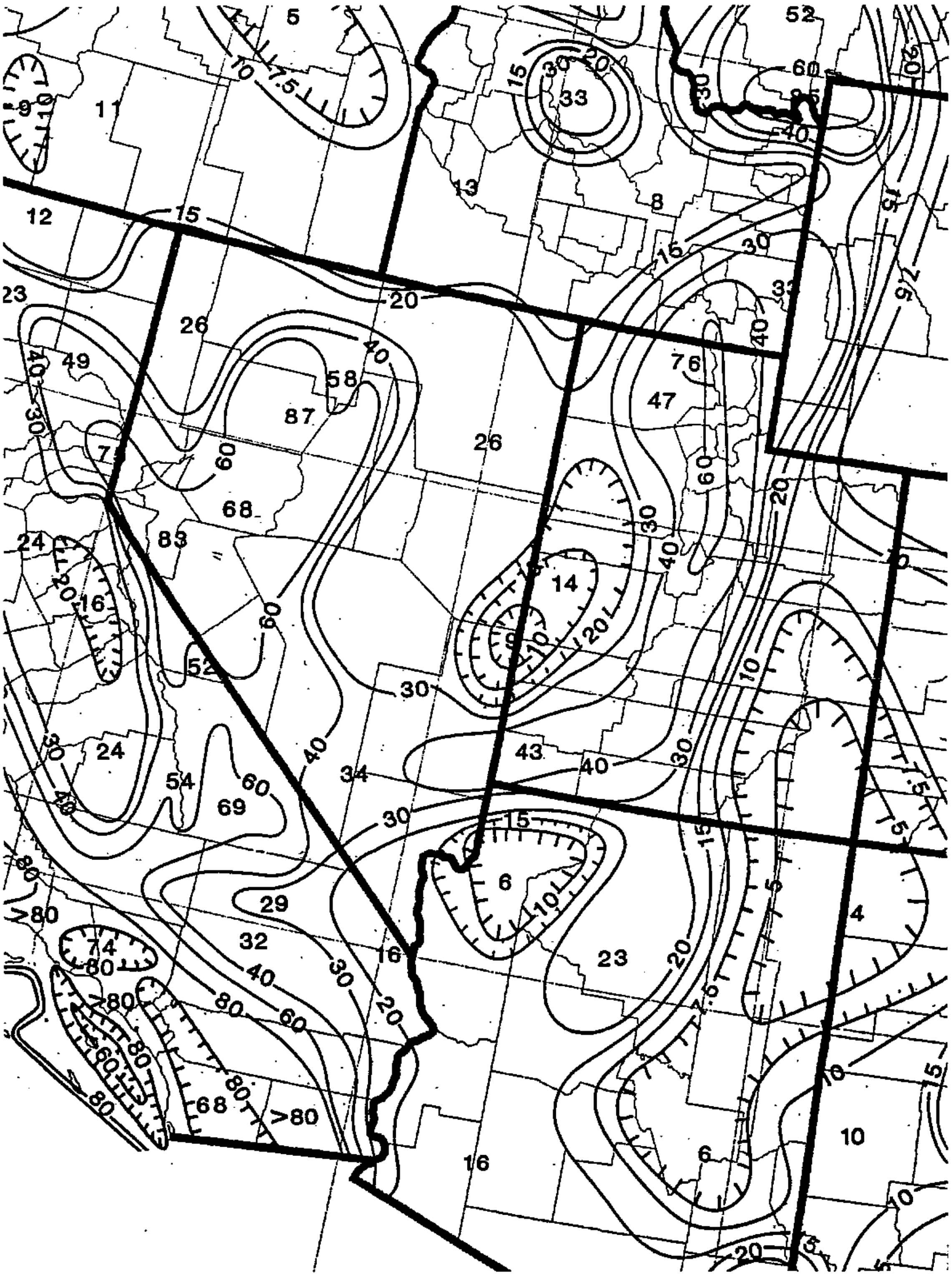


Figure 7: Probabilistic peak horizontal acceleration with a 10% chance of exceedence in 250 years for Utah and portions of the western U.S. (Algermissen et. al., 1990)

values in table 6. An example of how this seismic evaluation was implemented in Utah is illustrated by evaluation of the Interstate and Federal highways in the eastern part of the State (I-70, and Hwys. 191, 666, 40, 50 and 6 east of the 111° meridian). Figure 5 shows maximum magnitudes of 5.5 and 6.0 for the entire section of the State east of about the 111° meridian. Algermissen (1990) shows a_{max} contours for the same area ranging from less than 0.05 g to about 0.1 g with 90 percent probability of not being exceeded in 250 years. This combination of acceleration and magnitude when compared with the data in table 6 result in a low to "very low hazard". Thus all the bridge sites along these routes are classified as low hazard and low priority for future investigation. To check the validity of the seismic evaluation a site-specific analysis was conducted for I-70 crossing the Green River. A magnitude of 6.1 and a maximum acceleration of 0.1g were used in the analysis. The factor of safety against liquefaction for the site was calculated to be 6.2 or greater, verifying the very low hazard at the site.

3.4 Water Table Analysis

Groundwater depth in a region provides useful criteria for rapid identification of nonliquefiable sites. As indicated in the screening guide (Youd, 1998), liquefaction only occurs in saturated soils. Because granular soils usually become older and denser with depth, deeper saturated soils generally have lower liquefaction susceptibility. Table 7 lists water table depths below which liquefaction should not occur in alluvial soils. When determining the depth of the water table in areas where the water table fluctuates, a seasonal high level should be used. Youd (1998) states that an average high level over a 20-year period would generally be an adequately conservative value. In areas where the water table is consistently deep with only occasional rises when dry creek beds or arroyos sporadically flood, the long-term depth is adequate.

Table 7: Relative liquefaction susceptibility as a function of groundwater table depth (after Youd, 1998)

Groundwater Table Depth	Relative Liquefaction Susceptibility
< 3 m	Very High
3 m to 6 m	High
6 m to10 m	Moderate
10 m to 15 m	Low
> 15 m	Very Low

Depths to groundwater can be found in a variety of sources. Typical sources of water table depths are foundation investigations, well logs, and local and regional groundwater maps. A hydrogeologist familiar with the area might also provide useful information. The most reliable data should be used where available, but less reliable sources may be used when conservative assumptions are applied.

Example of Use of Groundwater Information to Screen Sites

Two principal sources of groundwater data were used for screening bridge sites in Utah. These sources were a map of shallow groundwater depths compiled by Hecker and others (1988), and water table measurements taken directly from borehole logs. The shallow groundwater map delineates areas where the groundwater is at depths less than 9 meters. This map was used to screen sites in areas characterized by peak accelerations and earthquake magnitudes that could cause liquefaction. Areas outside of the Wasatch Front (areas characterized by lower earthquake magnitudes and lower peak horizontal accelerations) with groundwater depths persistently deeper than 9 m were classified as low liquefaction hazard and low priority for further investigation.

Two examples of how this groundwater map was used are the I-15 bridge over the "I" line (station 711+80) between Fillmore and the Juab County line and I-70 over the South Salt Wash. The first site, the "I" line bridge over I-15, is located in an area that has a layer of topsoil ranging from 6.7 m to 14.3 m thick over conglomerate bedrock. The groundwater map by Hecker and others (1988) shows a water table depth greater than 9 m in the bridge area. Because of the depth of the groundwater, this site was classed as low hazard and low priority for further study. A borehole log from the site (figure 8), shows a possible layer of groundwater about 0.4 m to 1.3 m thick perched on the bedrock. That water may have been trapped drilling fluid from the drilling operation, but to verify that the site would not liquefy, the borehole data were analyzed using the simplified procedure. This analysis (results listed in appendix A) confirmed that the soils beneath the possibly perched water table are nonliquefiable.

The second site is the I-70 bridge over the South Salt Wash (approximately 90 km east of Salina) which is located in an area of substantial seismicity. The map by Algermissen (1990), figure 7, indicates a_{max} of 0.3 g for the area with 90 percent probability of not being exceeded in 250 years. The map created with Hanson and Perkins data, figure 5, indicates that an earthquake of magnitude 6.0 to 6.5 is possible. This magnitude and acceleration, when compared with table 6, indicate a moderate liquefaction potential. Because the water table, however, is mapped as deeper than 9 m, the hazard of liquefaction is low. This site was also analyzed using the site-specific analysis to confirm the assumptions and it was also determined that the sediments are too dense to liquefy (appendix A).

The second useful source of groundwater information was borehole logs from foundation investigations. In many instances, these logs were more useful than the shallow groundwater map because of the site-specific water table measurements. Many sites that could not be classed as low hazard using the groundwater map were classed as low hazard and low priority for future study due to the deep groundwater table shown on the borehole logs. Some examples of deep water tables are the Aragonite (Low) and Clive interchanges along I-80 in Tooele County. Both of these sites are underlain by granular soils, but as noted in figure 9, the drill holes at the Aragonite (Low) site reached 21 m without encountering groundwater. Figure 10 shows logs from the Clive site where holes reached 10.7 m without encountering ground water. For site-specific review of borehole logs, if the water table was not intercepted by drillers or was deeper than 15 m, the site was classed as low liquefaction hazard and low priority for further study.

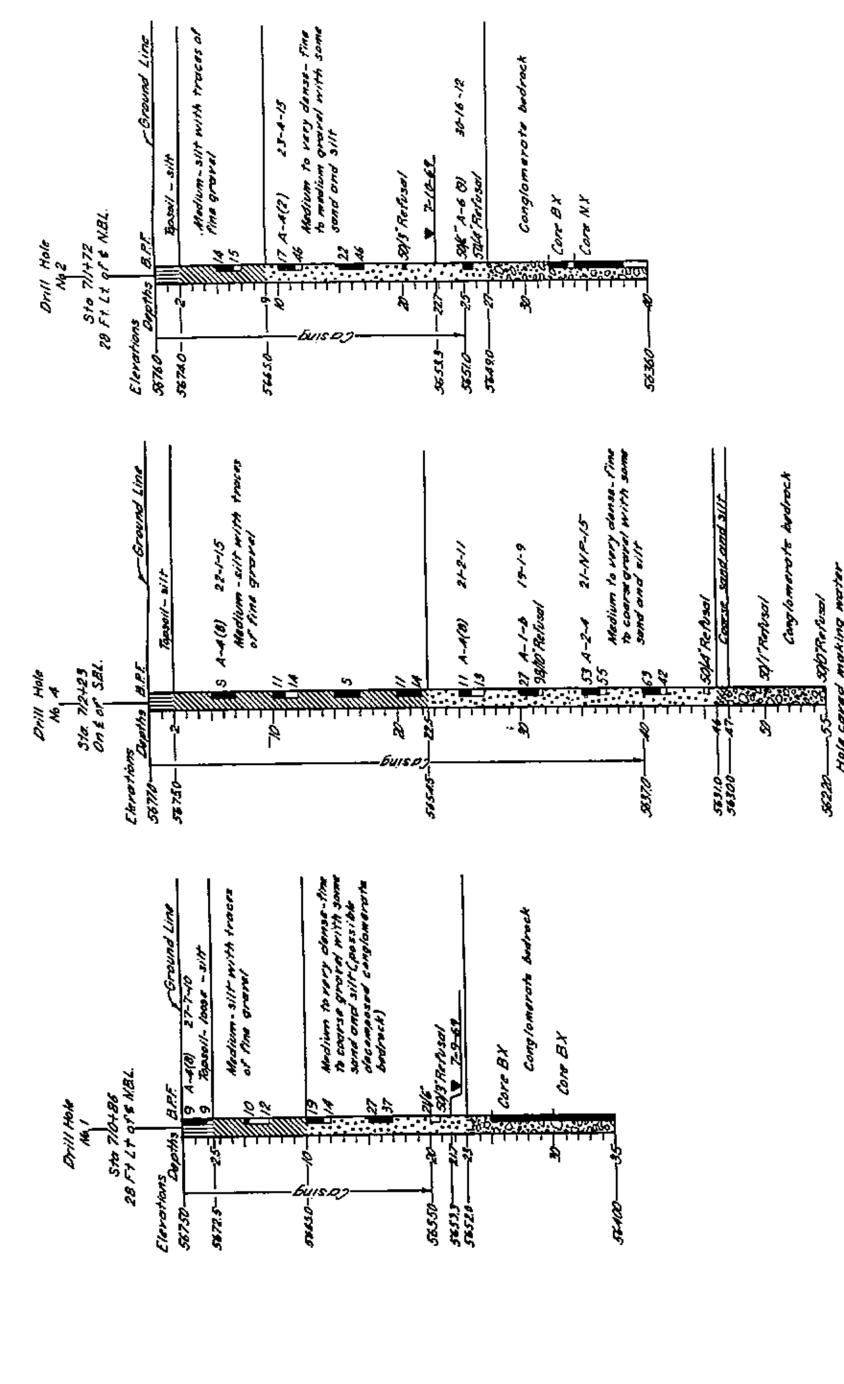
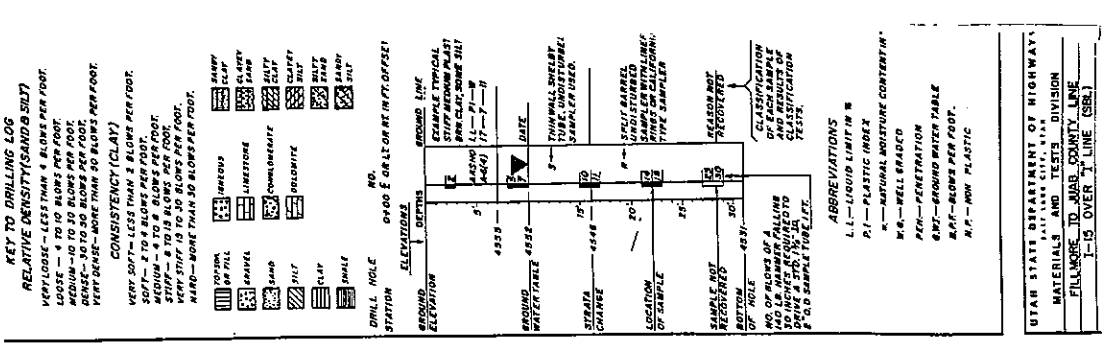



Figure 8: Borehole logs for the "I" line bridge site over I-15 showing groundwater levels

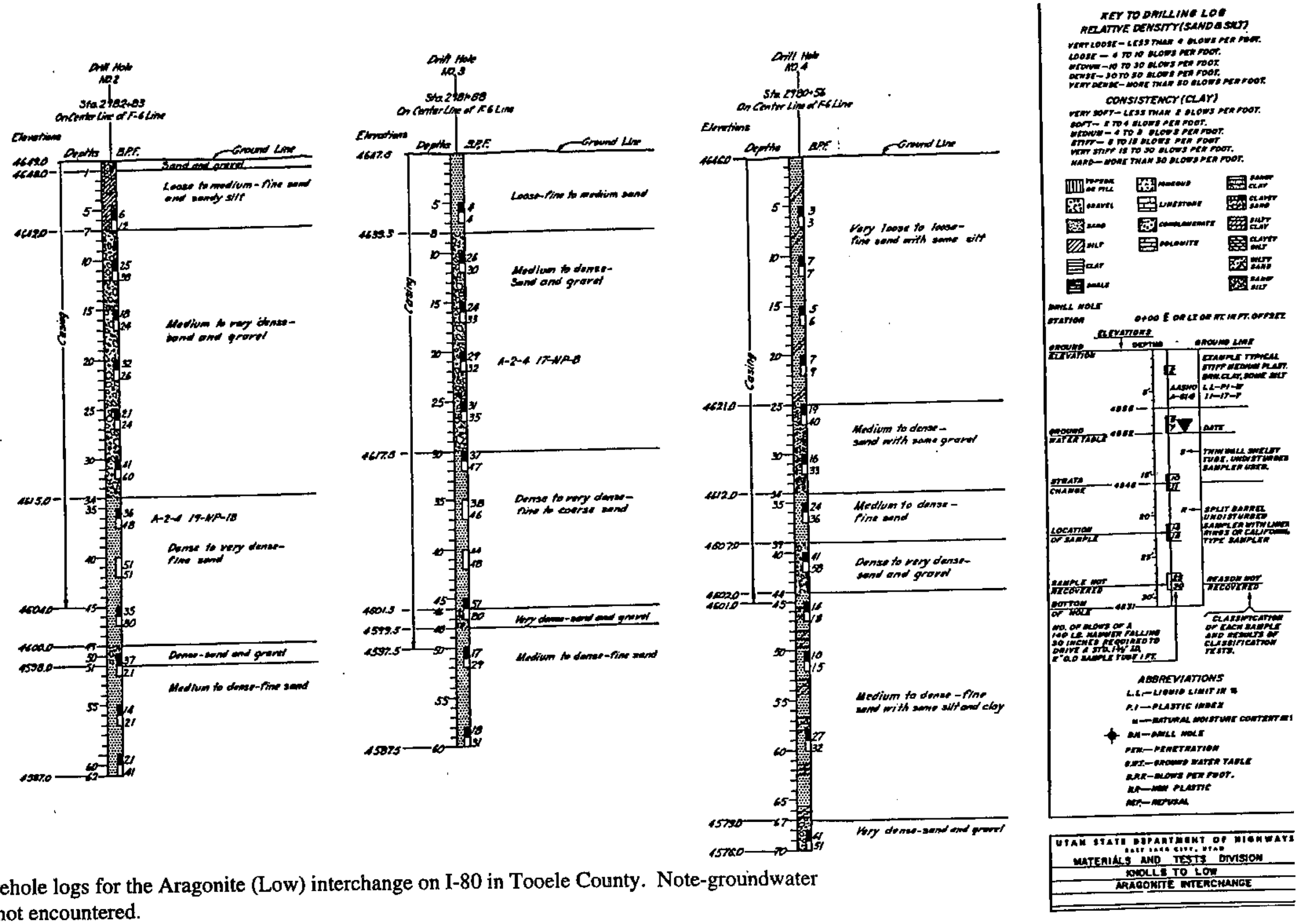


Figure 9: Borehole logs for the Aragonite (Low) interchange on I-80 in Tooele County. Note-groundwater was not encountered.

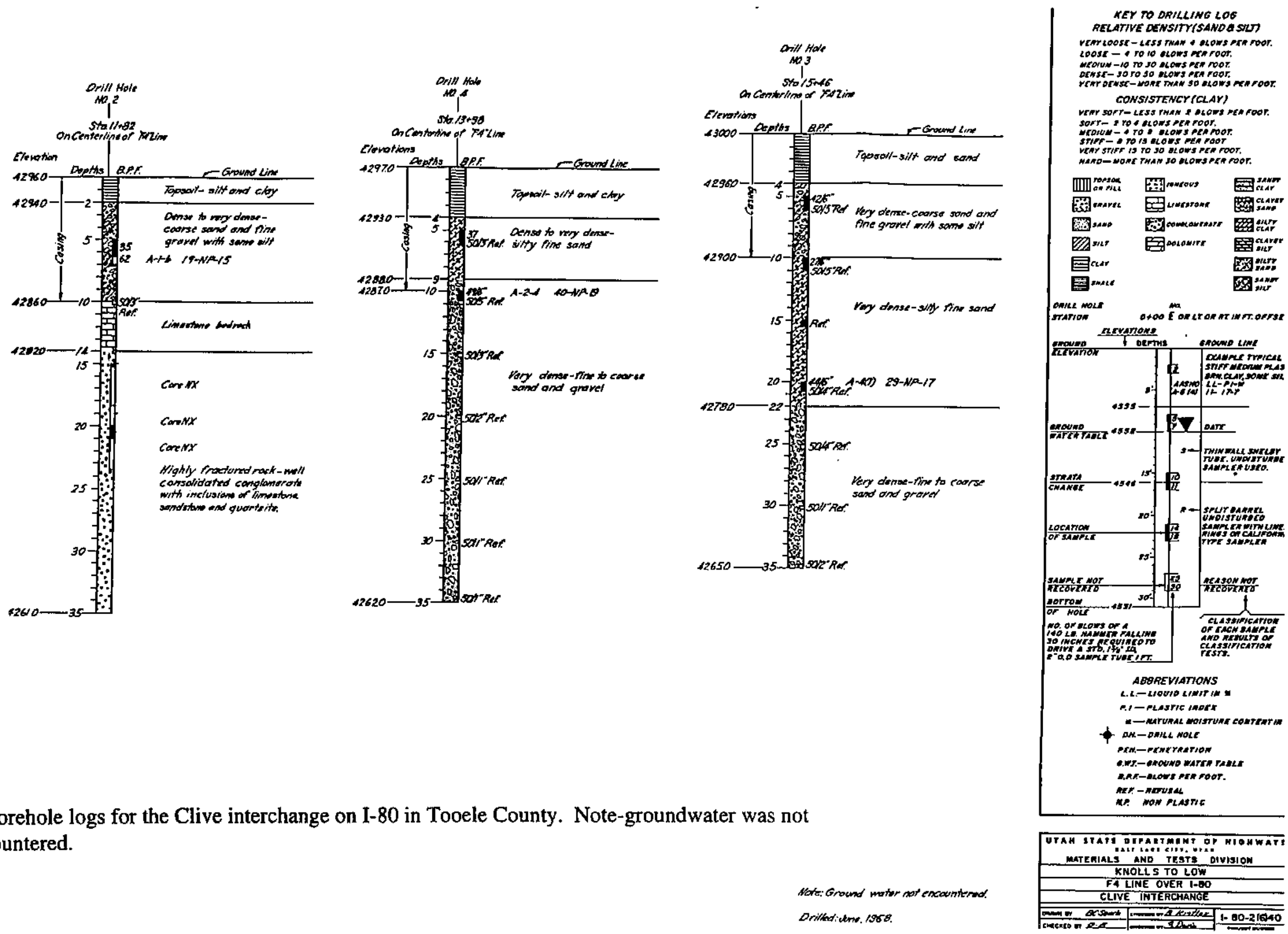


Figure 10: Borehole logs for the Clive interchange on I-80 in Tooele County. Note-groundwater was not encountered.

CHAPTER 4 - SITE-SPECIFIC EVALUATION

All bridge sites classed as possibly susceptible to liquefaction from the regional evaluations were further evaluated using site-specific procedures. These analyses required information from foundation reports, including penetration resistance, grain-size data, Atterberg limits, and stratigraphic cross sections. Where the required data were inadequate or unavailable, which was the case for nearly all older (pre-1960) bridges in the Federal and State highway systems, the site was classed as Priority I if the site is near a river or other water crossing or near a steep slope or high (greater than 5 m) embankment. If remote from these features, the site was classified as Priority III, indicating insufficient information of a site specific analysis, but sufficiently low ground failure hazard to warrant a low priority for further investigation. Where adequate site information was available, the following analyses were applied.

4.1 Screening for Extra Sensitive Clays

A phenomenon related to liquefaction is loss of strength due to cyclic straining of sensitive clays. Liquefaction is cause by consolidation of granular soils due to cyclic shear deformation during earthquake shaking. Sensitive clays, however, are fine-grained soils with a flocculated or "cardhouse" structure, which lose strength when remolded by cyclic deformation. These clays are rare because they are most commonly formed when percolating fresh groundwater leaches salt from soils deposited in a marine or saline lacustrine environment. Youd (1998) classifies extra sensitive soils as soft, low-plasticity clays with a liquidity index (LI) greater than 0.6, sensitivities greater than 4, a liquid limit (LL) less than 40 percent, a moisture content (w) greater than 0.9 times the liquid limit, and an $(N_1)_{60}$ less than or equal to 5 blows per 30 cm or a corrected cone penetration tip resistance less than 1 Mpa. The only soils that meet all of these requirements are Unified Soil Classification System (USCS) classifications CL and ML and AASHTO classifications A-2-4, A-2-6, A-4, and A-6. If the soil does not meet *all* of the above requirements it is classed as nonsensitive and the screening guide can be followed to the next step (Youd, 1998).

Example of Sensitive Soil Screen at Two Sites

Because the environmental conditions required for deposition of sensitive soils are rare, extra sensitive clays are not likely to occur in most parts of the country. However, a large part of Utah was once covered by Lake Bonneville, a large pre-historic lake that was saline in its later stages. The lake extended from Millard County in the south to the Idaho border in the north and from the Wasatch Front on the east to Nevada on the west. Most interstate routes in northern Utah lie on Lake Bonneville sediments that could contain sensitive layers due to the saline depositional environment created by the lake.

Possibly sensitive clays were identified beneath several bridge sites near the Great Salt Lake in the Bonneville Salt Flats. For example, the foundation investigation for the bridge at the AB line near the Solar Salt Plant west of Salt Lake City on I-80 contains the following information (see figure 11). Three boreholes at the site penetrated a soft clay layer between

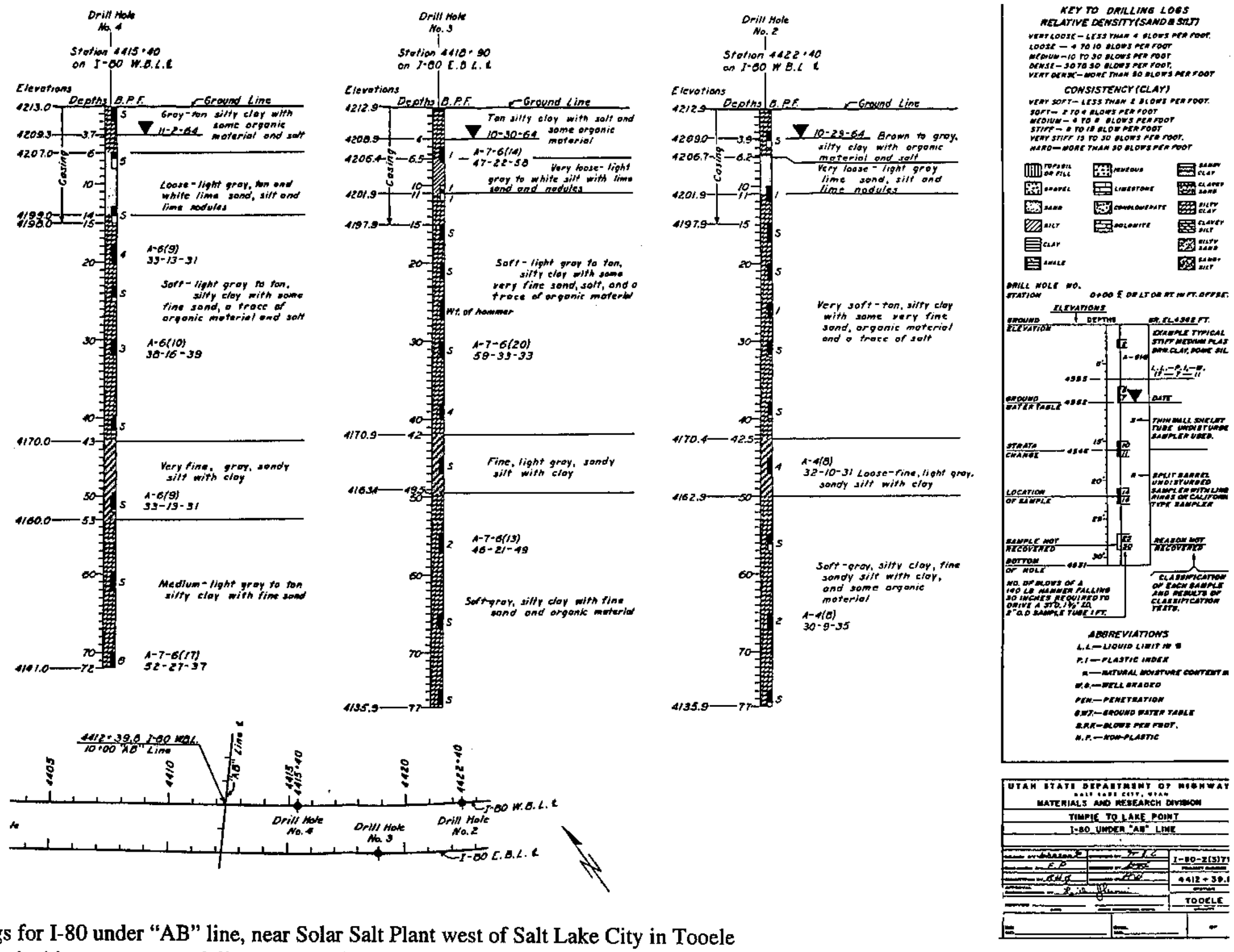


Figure 11: Borehole logs for I-80 under "AB" line, near Solar Salt Plant west of Salt Lake City in Tooele County. Drill hole 4 intersects potentially sensitive clays.

depths of 4 m and 13 m. The average corrected standard penetration blow count, $(N_1)_{60}$, resistances in the layer was 5.0 blows per foot (300 mm). These blow counts indicate a marginal likelihood that the layer is sensitive. Liquid limits determined from three soil samples taken from these layers were 33 percent, 38 percent and 59 percent, respectively. Plasticity indexes for the same samples were 13 percent, 16 percent and 33 percent and the respective moisture contents were 31 percent, 39 percent and 33 percent. These data yield liquidity indexes of 0.85, 1.07 and 0.27. The soils classify as AASHTO soil types A-6 and A-7. Thus, two of these the samples indicate that the clay could be sensitive. To be conservative, the site was classed as potentially sensitive, but classified as a Priority II for further investigation because of the flat terrain surrounding the site. Even if strength loss should occur in the clay, there is little potential for damaging ground displacements at this site. Additional tests to verify whether these soils are sensitive should include strength and sensitivity tests on tube samples extracted from the possibly sensitive layer. A 1-m to 2-m thick "lime sand" overlying the possibly sensitive clay layer was determined to be liquefiable using penetration resistance procedures, further warranting the Priority II classification for the site.

Another site with potentially extra sensitive soils is the bridge at 300 North and I-15 in Ogden. Drill hole 3 (figure 12) from that site shows a layer of clay from 7.6 m to 20.4 m deep. Tests on a sample taken at 10.7 m indicate an A-6 soil with an LL of 37 percent, a PI of 14 percent, a w of 34.6 percent, an LI of 0.83, and an $(N_1)_{60}$ of 2.7. This site should be investigated further because the soil meets the criteria for sensitive clay. In addition to screening the clays for sensitivity, the other soils in the profile should be checked for susceptibility to liquefaction.

4.2 Soil Classification Evaluation

As mentioned in section 3.2, liquefaction does not occur in all soils, but rather in a narrow range of classifications. Liquefaction is generally restricted to coarse-grained soils (silts, sands and gravels) that are sufficiently loose and uncemented that tend to compact during seismic shaking. In undrained or poorly drained soils, the tendency to compact leads to increased pore pressures that may ultimately generate a liquefied condition. Clay bonding between particles inhibits seismic compaction of fine-grained and cohesive coarse-grained soils, preventing compaction and pore pressure generation. Based on reported behavior of cohesive soils in areas of strong ground shaking, primarily from China, Seed and Idriss (1982) developed the following criteria to differentiate between liquefiable and non-liquefiable soils:

- clay content (minus 0.005 mm) less than 15 percent ($CC < 15$ percent)
- liquid limit less than 35% ($LL < 35\%$)
- moisture content greater than 0.9 times the LL ($w > 0.9LL$)

These criteria, commonly called "the Chinese criteria," are widely used by Geotechnical engineers. For a soil to be considered liquefiable, all three criteria must be satisfied.

Soils are typically classified by either the Unified Soil Classification System (USCS) or the AASHTO system using data from sieve analyses (gradation) and Atterberg limits. When

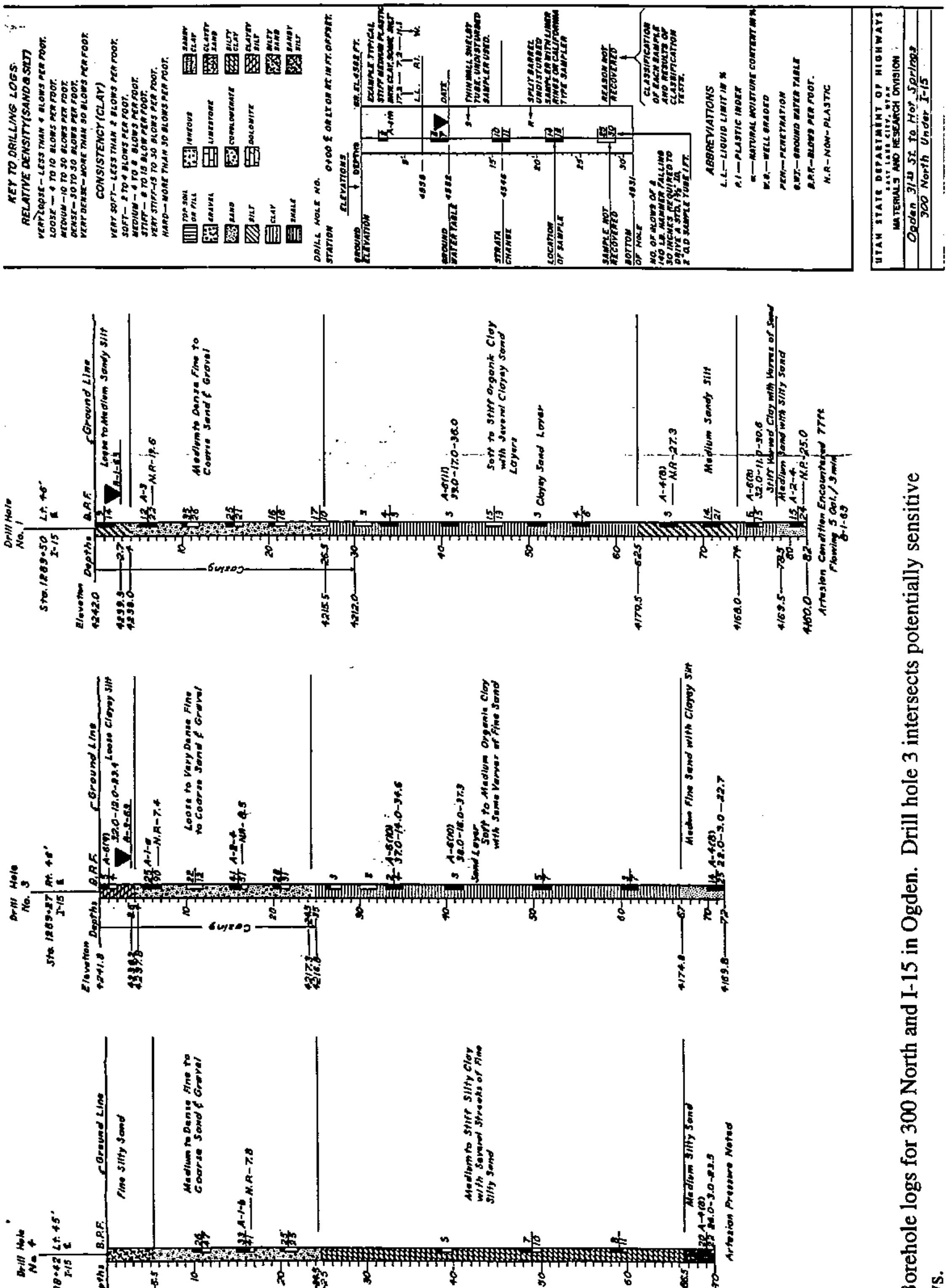


Figure 12: Borehole logs for 300 North and I-15 in Ogden. Drill hole 3 intersects potentially sensitive clays.

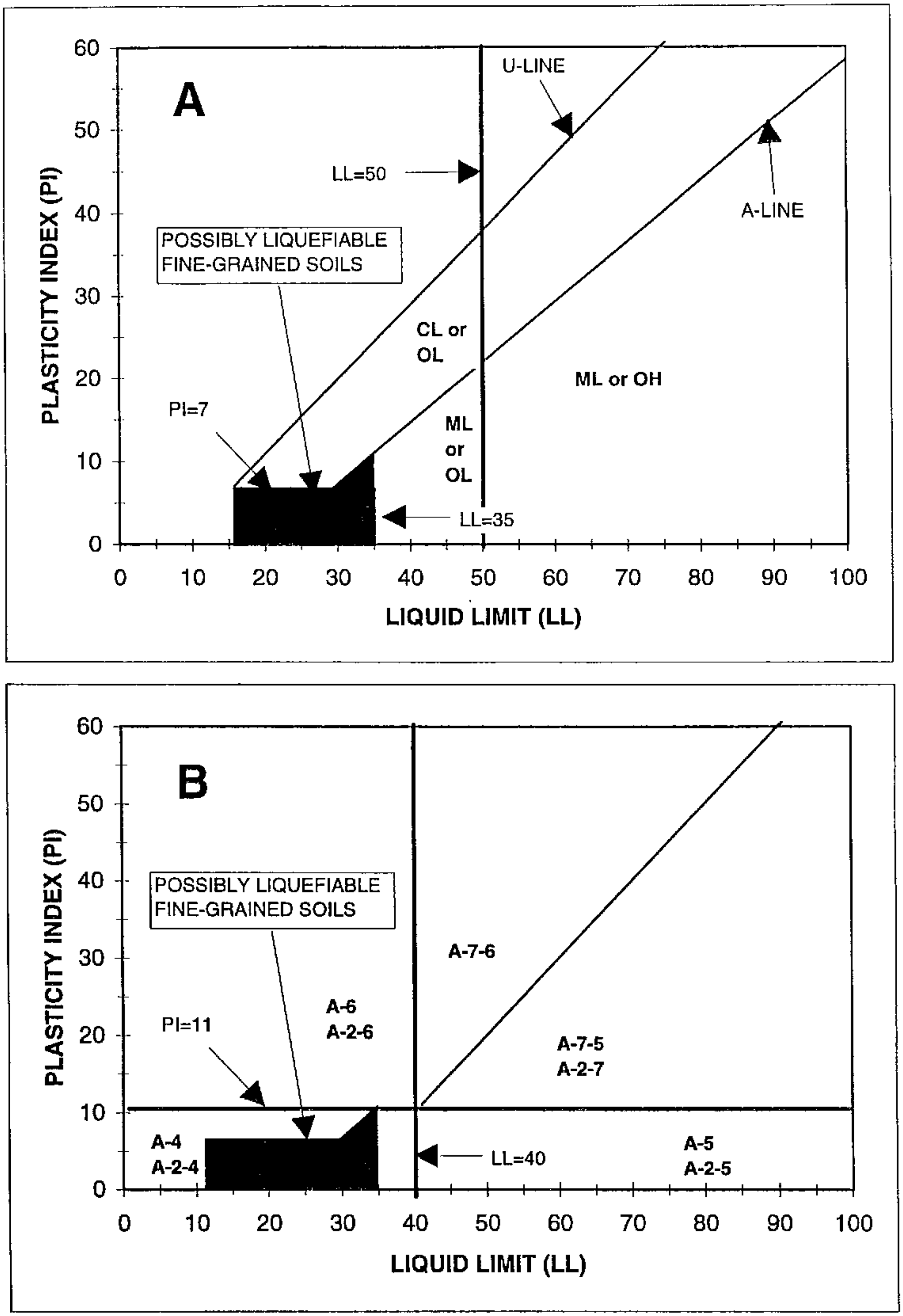


Figure 13: Chart showing (A) Unified Soil Classification System (USCS) and (B) AASHTO classification system for fine-grained soils with shaded region of chart indicative of liquefiable soils (after Youd, 1998).

using the USCS, the screening guide assumes that any soil which contains clay in the description (SC, GC, CL, or CH), has a clay content greater than 15 percent and will not liquefy. Soils with a dual group name (CL-ML, SM-SC, or GM-GC) could possibly liquefy due to the low clay content. Figure 13A shows USCS fine-grained soils with the shaded areas delineating soils which may liquefy.

The principal method of logging soil types on bridge foundation investigations is the AASHTO system. The only fine grained soils which can liquefy with AASHTO classifications are the A-2-4, and A-4 soils. Figure 13B shows the plasticity chart for the AASHTO classified soils with the shaded area transferred from figure 13A. This shows the soils with appreciable fines contents that may liquefy. As seen on this figure, not all A-2-4 and A-4 soils are liquefiable. Each of these soil types should be checked if data are available to determine if they fall into the shaded area. To be conservative, where Atterberg limits are not available, all A-2-4 and A-4 soils are considered potentially liquefiable.

Uncertainty may be encountered when word descriptions of soils are given instead of classification information such as Atterberg limits or other laboratory data. Visual descriptions without laboratory classifications do not provide precise information for determining whether silts or sandy silts have sufficient clay content to prevent liquefaction. To be conservative, when word descriptions are given instead of classifications or Atterberg limits, soils described with clay as the principal constituent are considered nonliquefiable (i.e. sandy clay, silty clay, or plastic clay). These soils are assumed to have clay contents greater than 15 percent. All other soils, including those where inadequate information is provided, are assumed to have clay contents less than 15 percent and thus possibly liquefiable.

Example of Screening Sites Based on Soil Type

Application of this step is illustrated by screening the bridge west of Tremonton at I-84 over US 30S at station 2633+74. This site is on late Pleistocene Lake Bonneville sediments. A copy of drill hole one, reproduced in figure 14, shows that the entire soil profile consists of A-6, A-7-5, and A-7-6 soils indicating nonliquefiable clays. This site was assigned a low hazard rating and a low priority for future study. Two additional sites were determined to have low liquefaction hazard because their foundations consist of nonliquefiable sediments. The two sites, I-80 under county road- station 68+60, and I-80 under access road- station 130+51, both near the Nevada border, have borehole logs (figures 15 and 16), which show that the soil profiles consist entirely of clays. Both sites are classed as low potential for liquefaction and low priority for future study.

Sometimes discrepancies exist between the word descriptions on borehole logs and the laboratory classifications. The word descriptions are primarily the driller's or field engineer's visual description of the soil rather than classifications based on laboratory data. When a discrepancy is found, the laboratory results are used because of their higher accuracy. As an example, a log from the Northbound I-215 bridge over the Jordan River, shows such a discrepancy. The field engineer listed the soil from 3.6 m to 8.5 m as being a silty clay (figure 17) while the lab data indicate this layer consists of clay *and silt* (AASHTO classifications A-6

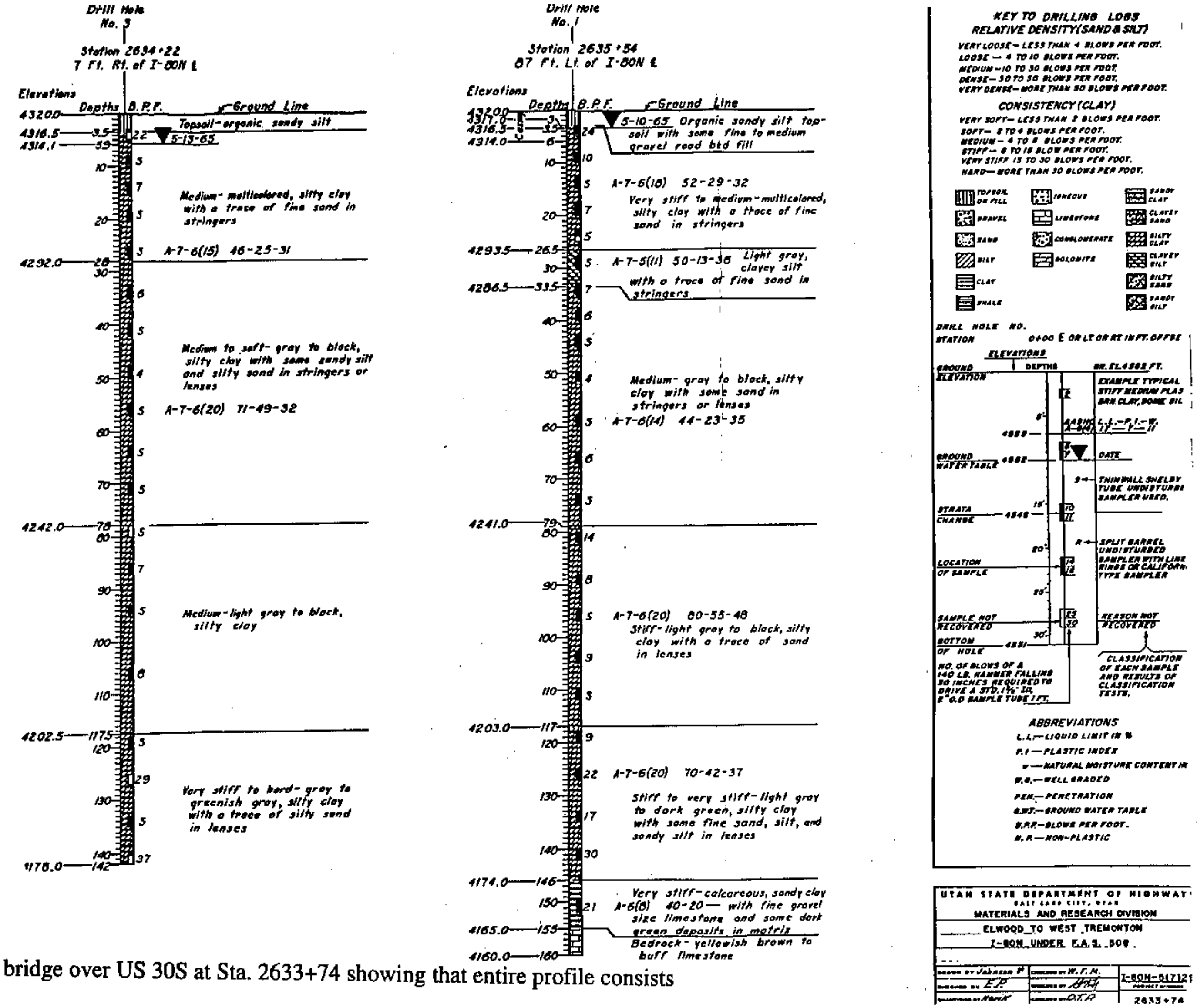


Figure 14: Borehole logs for I-84 bridge over US 30S at Sta. 2633+74 showing that entire profile consists of non-liquefiable clay

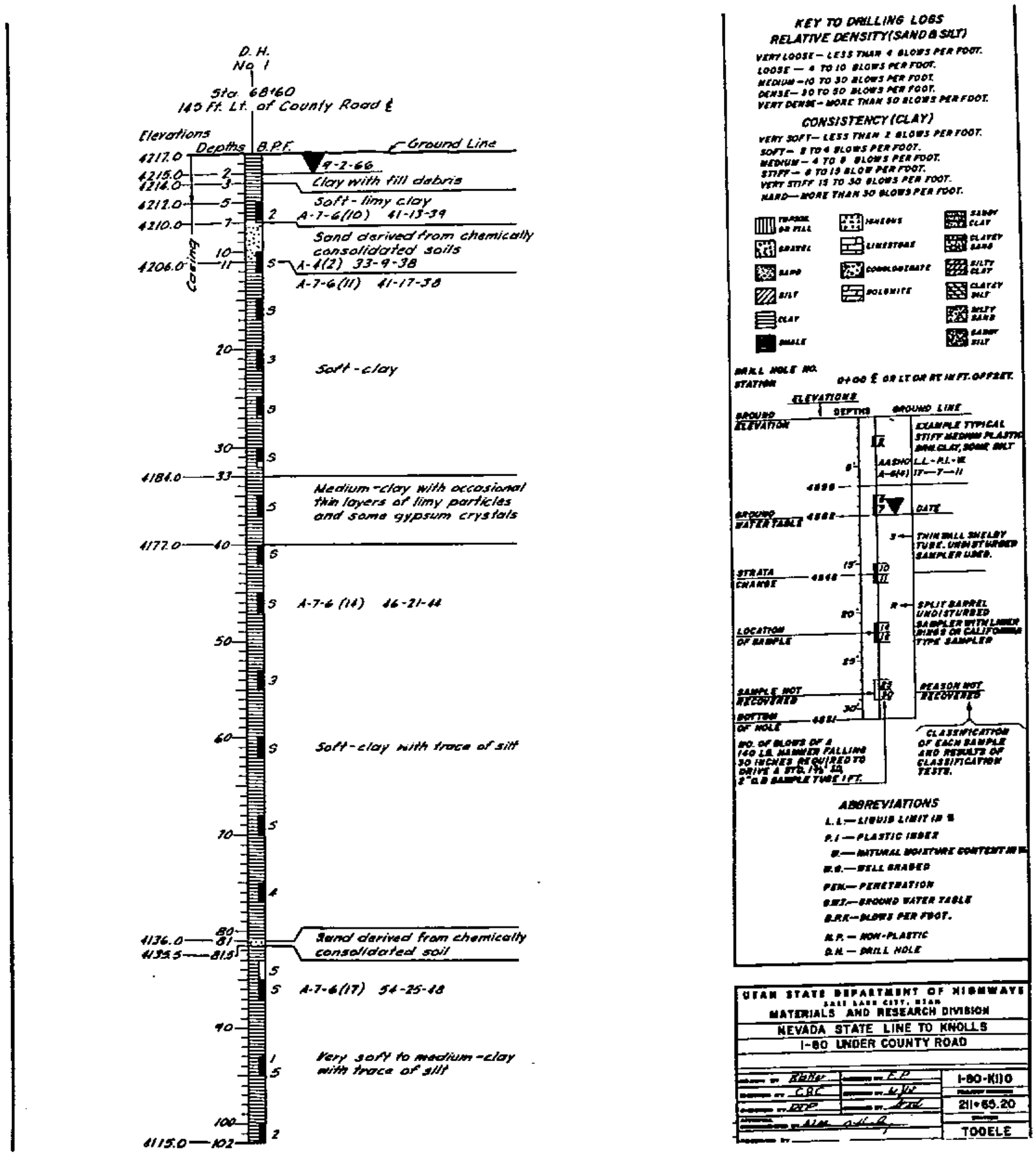
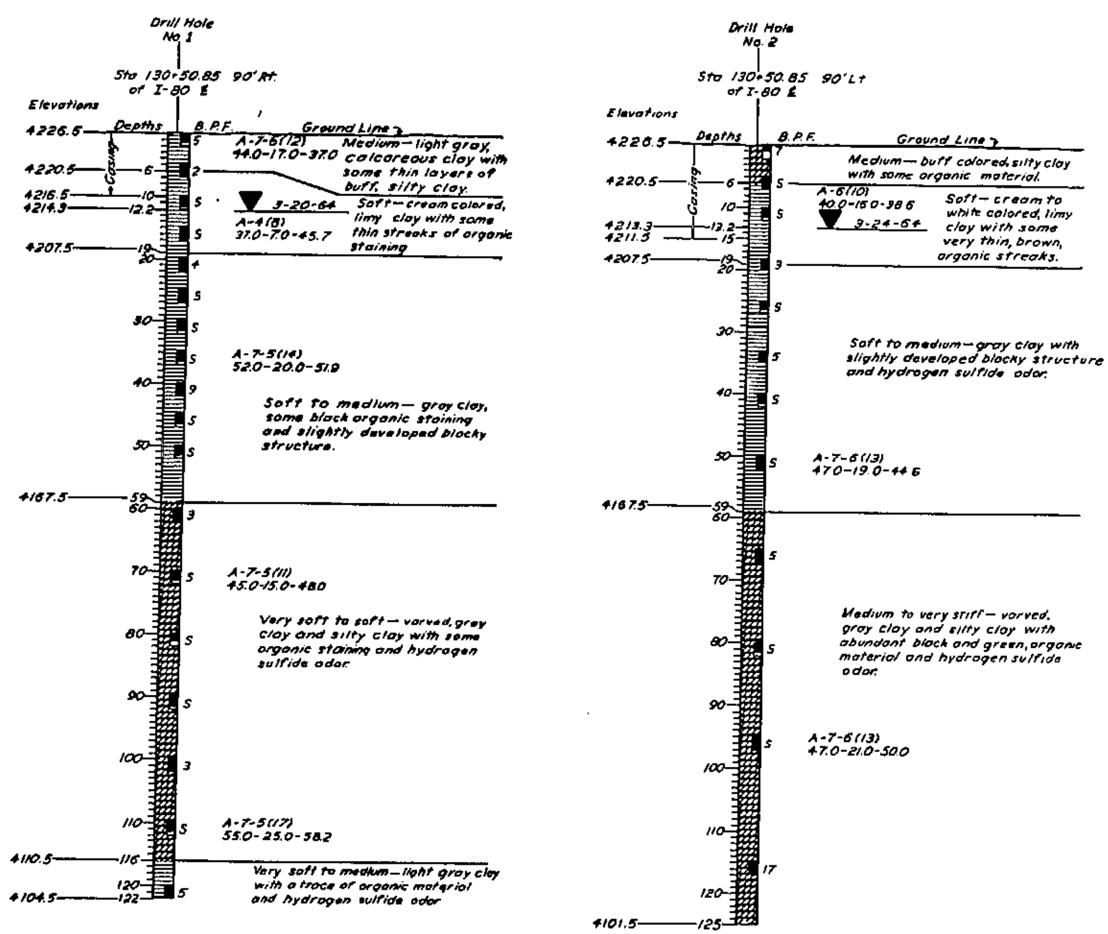


Figure 15: Borehole logs for I-80 under county road Sta. 68+60 (near Nevada border) showing that entire profile consists of non-liquefiable clay



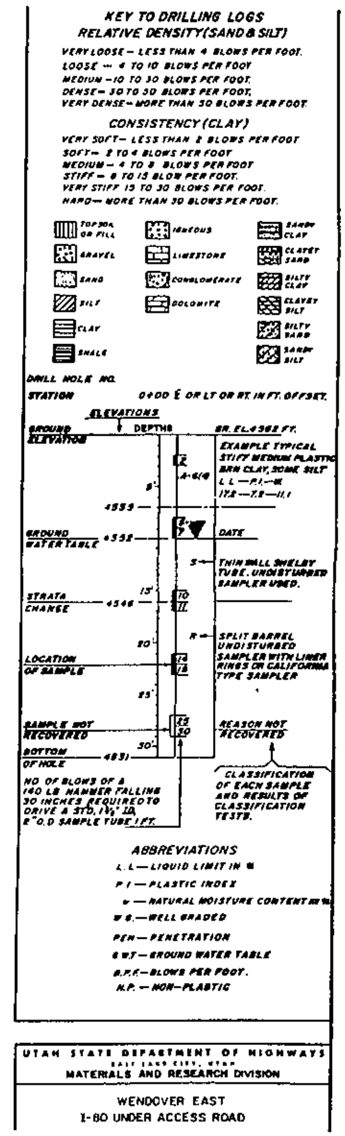


Figure 16: Borehole logs for I-80 under access road Sta. 130+51 (near Nevada border) showing that entire profile consists of non-liquefiable clay

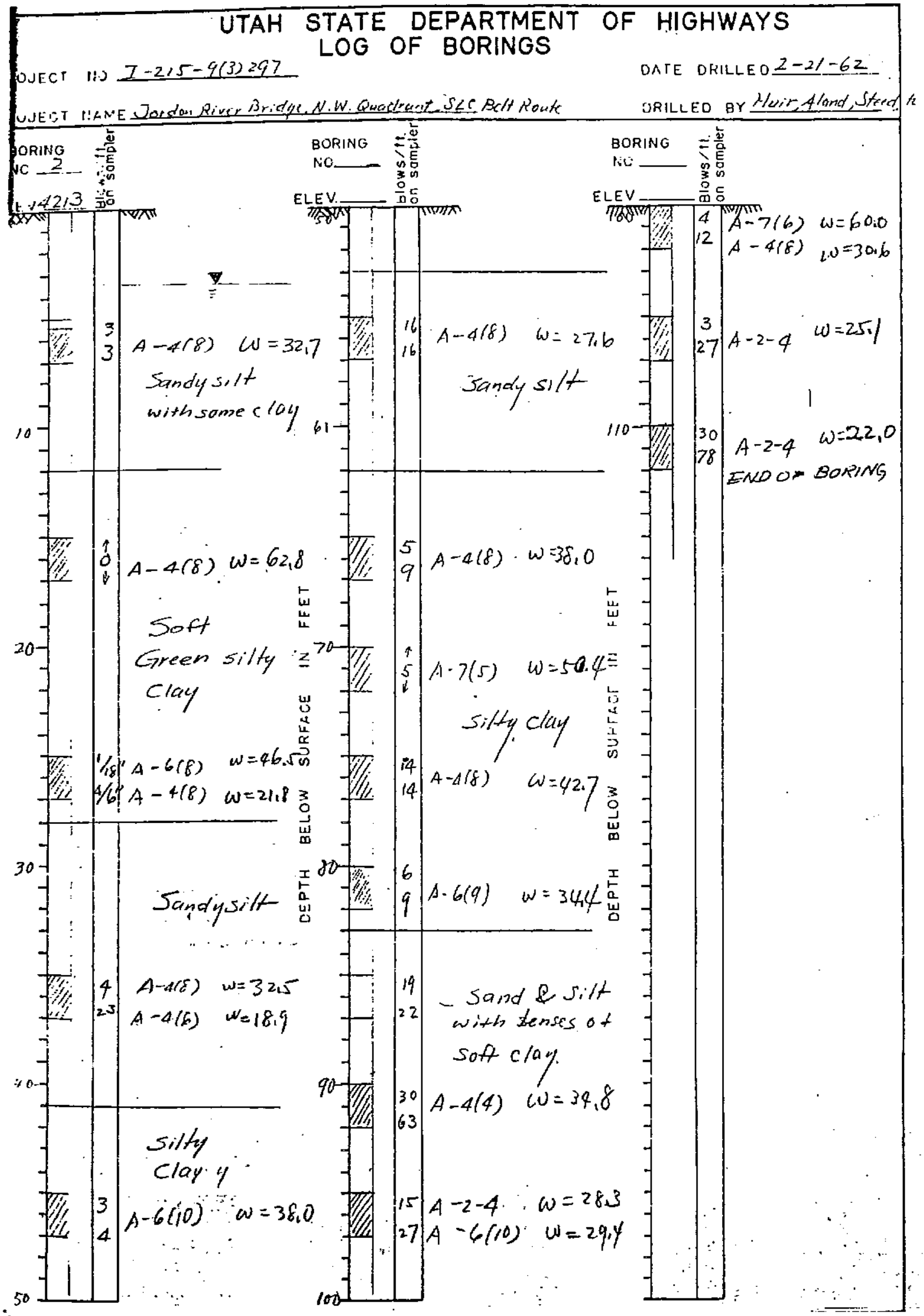


Figure 17: Borehole logs for I-215 bridge over the Jordan River (north) showing discrepancies between field engineer's visual classifications and laboratory classifications

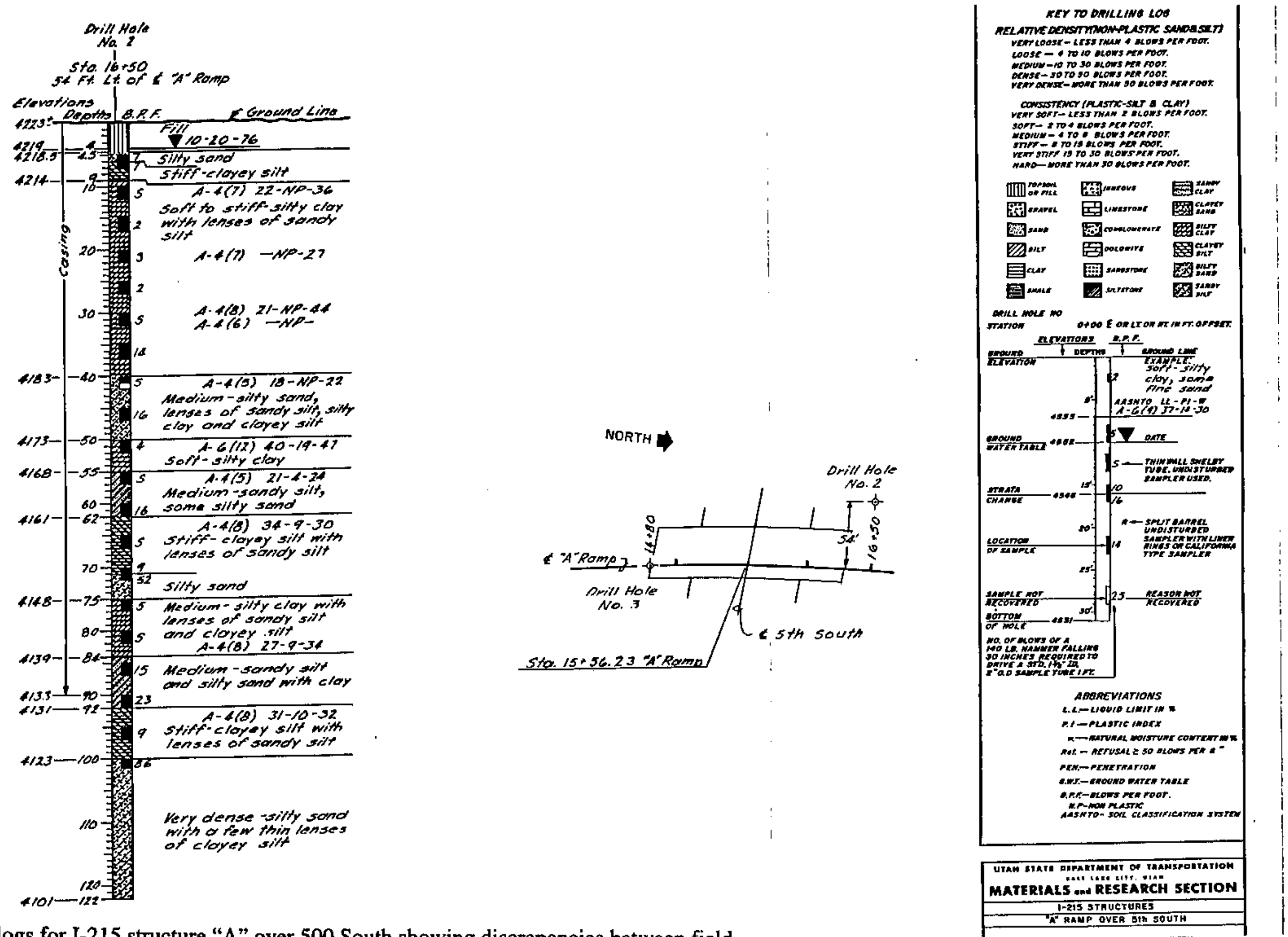


Figure 18: Borehole logs for I-215 structure "A" over 500 South showing discrepancies between field engineer's visual classifications and laboratory classifications

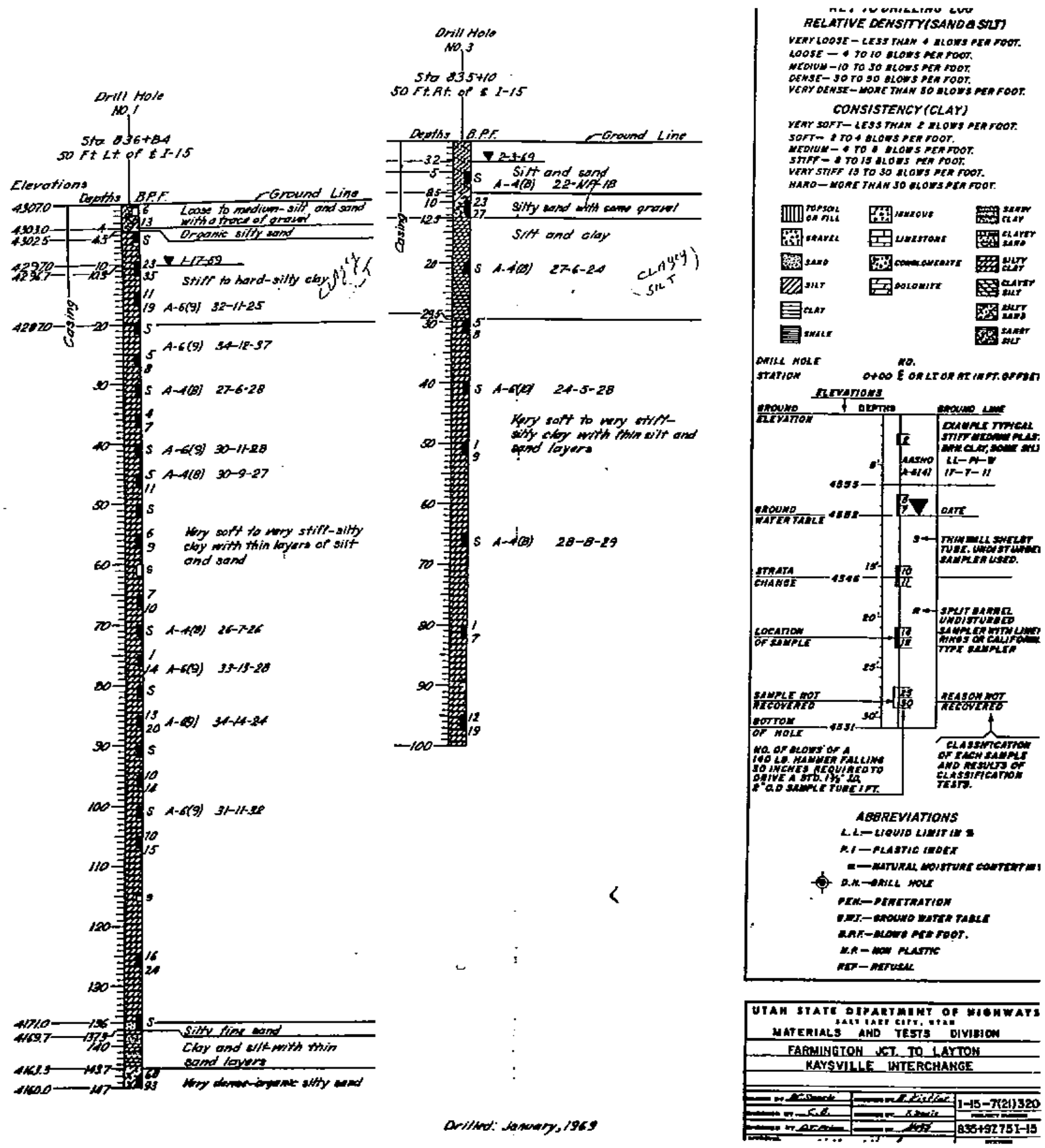


Figure 19: Borehole logs for Kaysville interchange showing discrepancies between field engineer's visual classifications and laboratory classifications

and A-4). The A-4 soils in the profile are potentially liquefiable so they were further analyzed. Another example of this discrepancy is demonstrated in figure 18 which shows drill hole two at the I-215 structure "A" ramp over 500 South in Salt Lake City. The log shows a single layer from 3 m to 12.2 m that is described as a "soft to stiff - silty clay with lenses of sandy silt". All four laboratory classifications in the layer, however, show the soil as being a non-plastic silt (A-4) which could possibly liquefy.

A related discrepancy in soil profiles is the tendency for field loggers to define thick layers of uniform soil, when in reality the layer may contain interbedded clays, silts, or sand. An example of this discrepancy is illustrated by figure 19 which shows drill hole one from the Kaysville interchange in Davis County. The log shows a single layer from 6.1 m to 41.4 m which is described as a "very soft to very stiff - silty clay with thin lenses of silt and sand". The laboratory classifications, however, indicate that this layer contains interbedded silts (A-4) and clays (A-6). The A-4 soils are potentially liquefiable.

4.3 Site-specific Calculations Using Standard Penetration Data

This step evaluates the liquefaction resistance of potentially liquefiable soils using the standard penetration resistance. The "simplified procedure" is the most widely used and accepted method for determining the liquefaction resistance (Seed et. al., 1985). The latest update of the simplified method, Robertson and Wride (1997), defines two variables used in calculating the factor of safety; the cyclic stress ratio generated by the earthquake (CSR), and the liquefaction resistance of the soil (CRR). The factor of safety against liquefaction is the ratio of the soil's resistance to liquefaction to the earthquake-induced stress driving the liquefaction, or:

$$FS = \left(\frac{CRR_{7.5}}{CSR_o}\right) MSF \tag{2}$$

where: MSF = magnitude scaling factor

$$CSR_{7.5} = \frac{\tau_{av}}{\sigma'_o} = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_o}{\sigma'_o}\right) r_d \tag{3}$$

where: τ_{av} = average cyclic shear stress generated by the earthquake
σ'_o = the pre-earthquake effective overburden stress
σ_o = total vertical stress
a_{max} = peak horizontal acceleration at ground surface
g = acceleration of gravity in same units as a_{max}
r_d = depth-related stress reduction factor, see figure 20

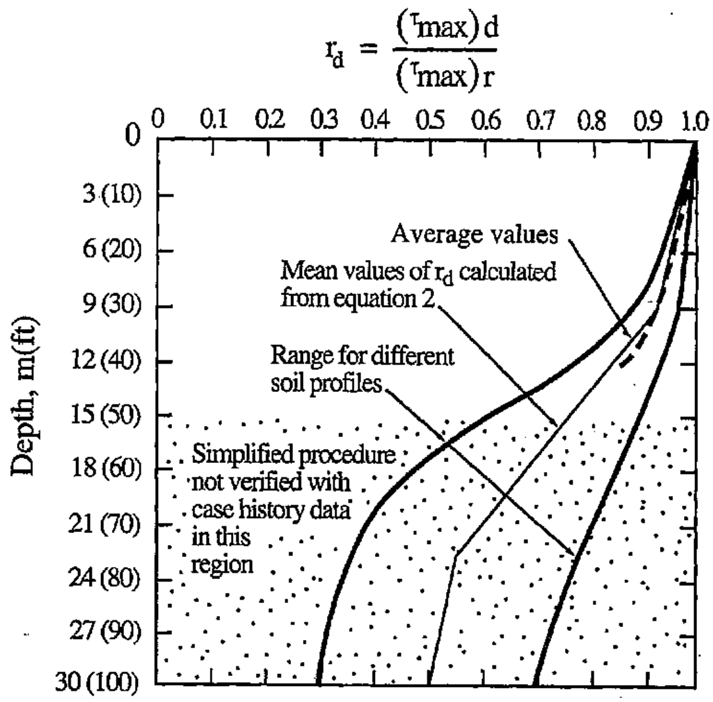


Figure 20: Range of r_d for different soil profiles and sediment depths. Developed by Seed and Idriss (1971) with Added Mean Value Lines (after Youd and Idriss, 1997)

The most common method for calculating the $CRR_{7.5}$ is the empirical relationship shown in figure 21. This figure was compiled by Seed and others (1985) and updated by Youd and Idriss (1997). Seed and others (1985) constructed the figure by plotting CSR versus $(N_1)_{60}$ values for sites that did or did not liquefy during magnitude 7.5 earthquakes. They drew conservative curves that separated data from sites that did or did not liquefy. These curves are used to calculate $CRR_{7.5}$ values with variances due to fines content. Figure 21 shows the three curves (35 percent, 15 percent, and ≤ 5 percent), used. Using these curves, $CRR_{7.5}$ can be found for any given $(N_1)_{60}$ by extending a line vertically from the $(N_1)_{60}$ abscissa to the corresponding fines content curve and then horizontally from that intersection over to the CRR & CSR axis. $CRR_{7.5}$, however, is only valid for a magnitude of 7.5 and must be corrected for other magnitudes using the magnitude scaling factors, MSF, shown in table 8.

The $(N_1)_{60}$ discussed above is calculated using the equation:

$$(N_1)_{60} = C_r C_n \left(\frac{ER_m}{60}\right) N_m \tag{4}$$

where: C_r = correction for drill rod length
C_n = correction factor for overburden pressure
ER_m = hammer efficiency for SPT tests
N_m = the measured standard penetration resistance

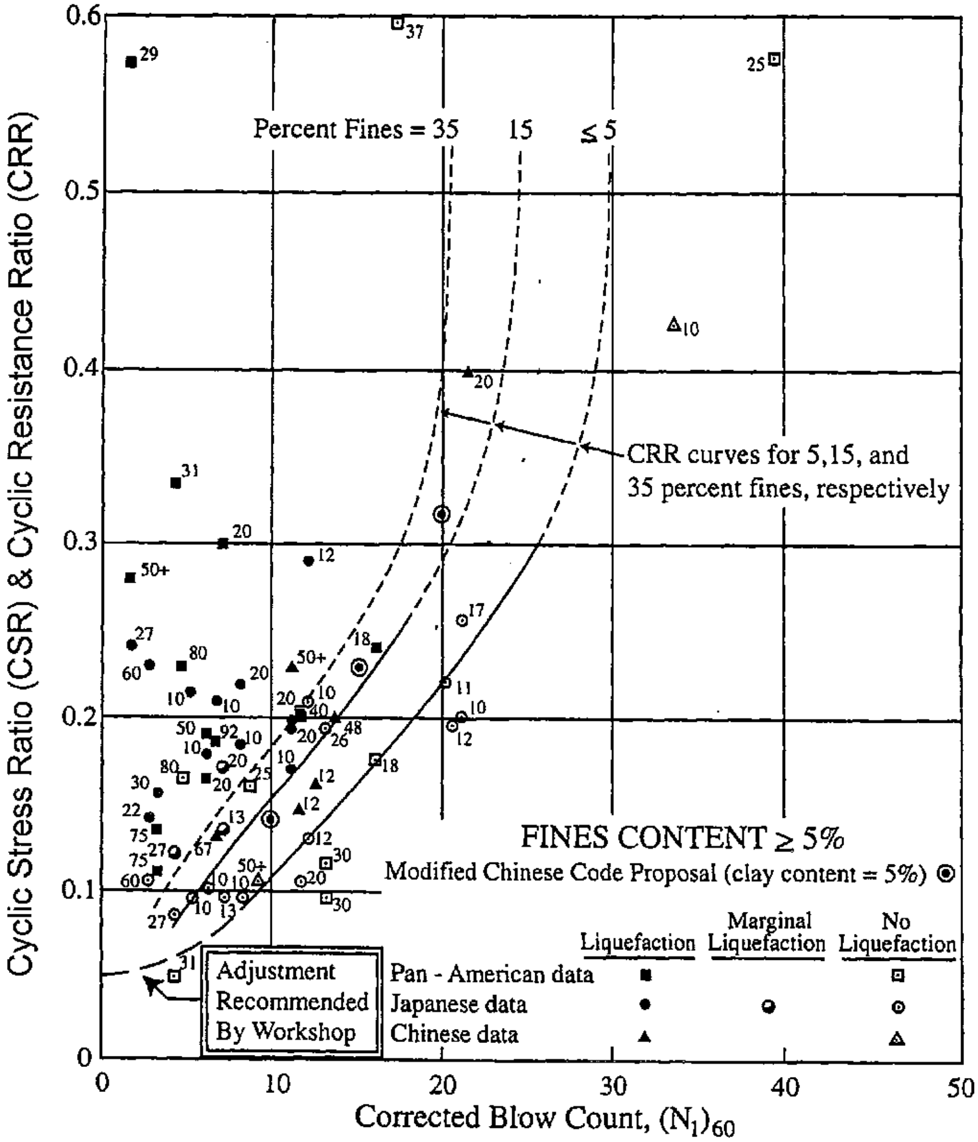


Figure 21: Relationship between CSR, CRR and $(N_1)_{60}$ for sands and silty sands and magnitude 7.5 earthquakes (after Youd and Idriss, 1997)

and:

$$C_n = \left(\frac{P_a}{\sigma'_o} \right)^{1/2} \tag{5}$$

where: The effective overburden stress, σ'_o, and the atmospheric pressure, P_a, are measured in the same units. The principal data needed to apply the simplified procedure are the standard penetration resistance, N_m, depth to the groundwater table, depth to the layer in question, the soil unit weight, fines contents, the magnitude, M, and the peak acceleration, a_{max}.

Table 8: Magnitude scaling factor values defined by various investigators (after Youd and Noble, 1997).

Mw (1)	Seed and Idriss (1982) (2)	Youd & Idriss (1997) (3)	Ambraseys (1988) (4)	Arango (1996) (5)	Arango (1996) (6)	Andurs and Stokoe (1997) (7)	p<20% (8)	Youd and Noble (1997) p<32% (9)	p<50% (10)
5.5	1.43	2.20	2.86	3.00	2.20	2.8	2.86	3.42	4.44
6.0	1.32	1.76	2.20	2.00	1.65	2.2	1.93	2.35	2.92
6.5	1.19	1.44	1.69	1.60	1.40	1.7	1.34	1.66	1.99
7.0	1.08	1.19	1.30	1.25	1.10	1.25	1.0	1.2	1.39
7.5	1.00	1.00	1.00	1.00	1.00	1.00			1.00
8.0	0.94	0.84	0.67	0.75	0.85	0.75?			0.73?
8.5	0.89	0.72	0.44			0.6?			0.56?

The standard penetration resistance, N_m, is usually recorded on borehole logs from the bridge foundation investigations. N_m is defined as the number of blows required to drive a standard split spoon sampler 300 mm, from 150 mm to 450 mm, with a standard hammer weighing 64 kg dropped from a standard height of 760 mm. Table 9 lists standard dimensions and requirements for the equipment used for standard penetrations.

The standard blow count is corrected to the equivalent blow count for 100 kPa of overburden pressure using equation (5). This correction factor, C_n, has a maximum value of two near the surface. Corrections are also required for short drill rod lengths and hammer energy. The factors C_r and ER_m in equation (4) are applied to make these corrections. For this study, a simple correction factor, C_r, of 0.75 was applied for depths less than 3 m (Seed et. al., 1985). The hammer energy ratio, ER_m, corrects the actual energy delivered to the drill rod by the hammer to a standard percentage of the theoretical maximum. Table 10 lists the energy ratios that should be used for various hammer types.

Seed and Harder (1990) suggest two additional correction factors, k_σ and k_α, for high overburden pressure and static shear stresses due to sloping ground, respectively. For the

Table 9: Recommended SPT procedure for use in liquefaction hazard assessment (after National Research Council, 1985).

Factor	Recommended Procedure
Borehole	102mm to 127mm diameter rotary borehole with bentonite drilling mud for borehole stability
Drill Bit	Upward deflection of drilling mud (tricone or baffled drag bit)
Sampler	OD = 50mm ID = 35mm constant (i.e. no room for liners in barrel)
Drill Rods	AW for depths less than 15 m N, BW, or NW for greater depths
Energy delivered to sampler (rod energy)	285 J (60% of theoretical maximum)
Blowcount Rate	30 to 40 blows per minute
Penetration Resistance Count	Measures over range of 150mm to 455mm of penetration into the ground

conditions at most interstate bridge sites (shallow liquefiable layers, less than 15 m deep and gentle slopes, less than 10 percent), these correction factors are near 1 and can be safely neglected.

To calculate the total and effective overburden stresses (σ_0 and σ'_0 respectively), depth to groundwater and unit weights of soil layers are needed. The groundwater table data are usually recorded on driller's logs. Sometimes soil unit weights are also recorded on these logs. For sites where the unit weights were not measured, Youd (1998) recommends using the following average values: $\gamma_{dry} = 17$ kN/m^3, $\gamma_{moist} = 19$ kN/m^3, $\gamma_{sat} = 21$ kN/m^3. These are conservative estimates of unit weights which will give higher, more conservative CSR values.

Table 10: Summary of Energy Ratios for SPT Procedures (after National Research Council, 1985)

Country	Hammer Type	Hammer Release	Estimated Rod Energy (percent)	Correction Factor for 60 Percent rod Energy
Japan	Donut	free-fall	78	78/60 = 1.3
	Donut	Rope and pulley with special throw release	67	67/60 = 1.12
United States	Safety	Rope and pulley	60	60/60 = 1.0
	Donut	Rope and pulley	45	45/60 = 0.75
Argentina	Donut	Rope and pulley	45	45/60 = 0.75
China	Donut	Free-fall	60	60/60 = 1.0
	Donut	Rope and pulley	50	50/60 = 0.83

A soils fines content is occasionally measured and recorded on borehole logs. This value is needed to select the appropriate curve from figure 21. Where fines contents are not measured, they may be estimated from soil classification information on borehole logs, Atterberg limits, or other available information on soil types. If no information is available, an assumption of the most susceptible soil type, clean sands, will produce conservative results.

Moment magnitude, M_w, should be used for liquefaction hazard calculations, but surface wave, M_s, or local, M_L, magnitudes may be used for magnitudes between 5.5 and 7.5. The preferred method for calculating a_{max}, are peak acceleration attenuation curves. For soft sites, response analyses using computer programs such as SHAKE or DESRA provide useful estimates. Estimates from regional or national probabilistic maps of a_{max}, as mentioned in section 3.3, may also be used.

The simplified procedure has only been validated for depths less than 15 m. Historically, few sites, if any, have liquefied at greater depths. As noted in appendix A, many borehole logs contain data for depths greater than 15 m. For this study, the simplified procedure was extrapolated to estimate liquefaction resistance of deeper layers at soft sites.

Example Calculation Using Standard Penetration Data

As outlined in section 3.1, the first screen applied to the I-15 corridor in Salt Lake County was a review of the liquefaction potential maps compiled for the area by Anderson and others (1994d). These maps indicate that almost the entire I-15 corridor through Salt Lake County lies in areas characterized by either high or moderate liquefaction hazard. As noted on the flow chart in figure 3, the screening of such sites proceeds directly to site-specific analyses.

The simplified procedure was applied to evaluate liquefaction resistance for each bridge site in the I-15 corridor in Salt Lake County. Equations (2), (3), and (4) and the curves in figure 21 were programmed into a database to simplify the calculations. The program calculates a factor of safety every 10 cm with linear interpolations between blow counts and near layer boundaries. Idriss' magnitude scaling factors, as listed in column 3 of table 8, were used in the initial screen to adjust for magnitude. To be conservative, layers with FS > 1.2 are classed as non-liquefiable, while those with FS ≤ 1.2 are considered liquefiable.

The example calculations are of the sand layer at a depth of 2.7 m to 4.0 m shown by borehole DH-15 at 600 South in Salt Lake City (figure 22). The CSR is calculated using equation (3).

$$CSR = 0.65(0.5g)\left(\frac{61kPa}{37kPa}\right)0.99 = 0.53 \qquad \text{from equation (3)}$$

where: $r_d = 0.99$ from figure 20

$\sigma_o = 61$ kPa at a depth of 3.0 m

$\sigma'_o = 37$ kPa at a depth of 3.0 m

$a_{max} = 0.5$ g

$M_w = 7.5$

Depth				Soil Classification			Gradation (%)						Atterberg Limits		
m	ft	Soil Profile	Method (blows/ft)	AASHTO	USCS	Soil Description	gravel	sand	fines (silt & clay)	Undrained Shear Strength (kPa) [tsf]	Dry Unit Wt. (kN/m^3) [pcf]	Moisture Content (%)	LL	PL	PI
0.0	0														
0.3	1				SW	sand and gravel									
0.6	2														
0.9	3														
1.2	4														
1.5	5					fine sandy									
1.8	6		SPT (11)	A-4 (8)	ML	silt w/ traces	2.5	22.3	75.2	65 [0.68]	16 [101]	23	NP	NA	NA
2.1	7					of clay									
2.4	8														
2.7	9														
3.0	10					sand w/									
3.4	11		SPT (15)		SM	some silt									
3.7	12														
4.0	13														
4.3	14														
4.6	15		Shelby												
4.9	16		Shelby							42 [0.44]	12 [77]	40			
5.2	17														
5.5	18					silty clay									
5.8	19														
6.1	20														
6.4	21		Shelby												
6.7	22		Shelby	A-6 (9)	CL		0	1.4	98.6		11.6 [74]	50.3	34	23	11
7.0	23														
7.3	24														
7.6	25		Shelby												
7.9	26		Shelby	A-6 (10)	CL		0	0.6	99.4	40 [0.42]	13 [83]	39	37	23	14
8.2	27														
8.5	28														
8.8	29														
9.1	30		Shelby												
9.4	31		Shelby		CL					43 [0.45]	12.9 [82]	34.5	36	21	15
9.8	32														
10.1	33														
10.4	34														
10.7	35														
11.0	36		SPT (5)			silty clay									
11.3	37														
11.6	38														
11.9	39														
12.2	40		Shelby												
12.5	41		Shelby	A-7-6 (17)	CH		0	0.5	99.5		11.3 [72]	45.5	50	24	26
12.8	42														
13.1	43					silty clay									
13.4	44														
13.7	45														
14.0	46														
14.3	47														
14.6	48		SPT (14)												
14.9	49														
15.2	50														
15.5	51					silty clay									
15.8	52														
16.2	53														
16.5	54														
16.8	55		Shelby												
17.1	56		Shelby	A-4 (7)	ML	sandy silt	0	28.1	71.9		16.4 [104.6]	19.6	21	16	2
17.4	57														
17.7	58														
18.0	59														
18.3	60		Shelby							84.5 [0.88]					
18.6	61		Shelby		CH					76.8 [0.8]	13.2 [83.8]	41.7	55	23	32
18.9	62														
19.2	63														
19.5	64					silty clay									
19.8	65		Shelby			w/ some									
20.1	66		Shelby	A-7-6 (20)	CH	clayey sand	0	2	98		11.7 [74.5]	42.4	59	25	34
20.4	67					lenses									
20.7	68														
21.0	69														
21.3	70														
21.6	71		SPT (12)												
21.9	72														
22.3	73														
22.6	74														
22.9	75														

Figure 22: Soil profile and laboratory data for 600 South offramp. Data compiled from boreholes DH-15 and DH-15A. (Gerber, 1995)

An acceleration of 0.5 g and a magnitude of 7.5 were used for all bride site analyses from Utah County, north to the Idaho border. In a parallel study, Gilstrap (section 4.4 of this report) uses a higher acceleration, a_{max} = 0.6 g, and a lower magnitude, M_w = 7.0. Gilstrap's values yield a much higher CSR of 0.633. This higher value is partially offset by a lower magnitude yielding a lower CRR and approximately equal factors of safety.

Next, $(N_1)_{60}$ is calculated for use in figure 21 to estimate the $CRR_{7.5}$. For the sand layers, an uncorrected blow count, N_m = 15, was recorded at a depth of 3.0 m. This value was assumed to characterize the entire layer. The ER_m estimated (none were measured) for the majority of analyses in Utah was 50 percent. Because the penetration test depth was close to 3 m, a C_r of 1.0 was assumed. Converting the effective stress to units of kPa for use in equation (5) yields σ'_o = 38 kPa. For $P_a \cong$ 100 kPa:

$$C_n = \left(\frac{100kPa}{38kPa}\right)^{1/2} = 1.62 \qquad \text{from equation (5)}$$

Plugging these values into equation (4) gives:

$$(N_1)_{60} = (1)(1.62)\left(\frac{50}{60}\right)(15) = 20 \text{ blows-per-foot} \qquad \text{from equation (4)}$$

The fines content for this soil sample was not measured, so a fines content of 35 percent was estimated from the visual classifications of silty sand. In this study, all soils visually classified as silty sands were assigned a fines content of 35 percent for liquefaction hazard calculations. Using $(N_1)_{60}$ = 20 and a fines content of 35 percent, a $CRR_{7.5}$ of 0.4 was estimated (figure 21). For a M_w of 7.5, the MSF is 1.0 and the factor of safety against liquefaction is:

$$\text{FS} = \left(\frac{0.4}{0.53}\right)(1) = 0.76 \qquad \text{from equation (2)}$$

Using the lower magnitude of M_w = 7.0 as utilized by Samuel Gilstrap (section 4.4), the magnitude scaling factor is MSF = 1.19 (Idriss) as listed in table 8. Using the Idriss MSF of 1.19, the factor of safety using the higher acceleration of a_{max} = 0.60 g and the lower magnitude of M_w = 7.0 is:

$$\text{FS} = \left(\frac{0.40}{0.63}\right)(1.19) = 0.75 \qquad \text{from equation (2)}$$

Note that the two analyses give almost identical factors of safety. Because these factors of safety are less than 1.2, the layer was classed as potentially liquefiable.

Because conservative assumptions were made, the analysis should classify more materials as liquefiable than may actually exist. Gilstrap made additional analyses using data from subsurface investigations for the reconstruction of I-15 in Salt Lake County where high-quality drilling and standardized testing techniques were applied. These additional analyses greatly reduce the degree of uncertainty in the hazard evaluations and also lead to about a 50 percent reduction in the amount of sediment classed as liquefiable (section 4.4)

4.4 Analyses Using Cone Penetration Data

Samuel D. Gilstrap, an undergraduate student assigned to assist with this analysis, conducted the analyses and prepared much of the text reported in this section. This assistance is gratefully acknowledged. Both standard (SPT) and cone (CPT) penetration data were used in an integrated analysis, hereafter referred to as the "integrated analysis". For this study, the integrated analysis utilized state-of-the-art liquefaction analysis procedures and high quality subsurface data from preliminary subsurface investigations for the reconstruction of I-15 in Salt Lake County.

4.4.1 Study Objective

The purpose of the integrated analysis was to reanalyze liquefaction hazard at bridge sites along the I-15 corridor using data from recent subsurface investigations. The previous evaluation was based on available, but incomplete soil data from the original foundation investigations for the freeway. This study utilized data from recent (1996) subsurface investigations in the I-15 corridor. Cone penetration tests (CPT) were conducted at most bridge sites. Standard penetration tests (SPT), with field calibrated energy ratios, were performed at every bridge site. Elevations, coordinate locations, and water table depths were measured at all CPT and SPT locations. Laboratory data included grain-size gradations, Atterberg limits, and natural moisture contents. These data were made available to this investigation by the various contractors.

For this analysis, Gilstrap applied state-of-the-art procedures developed by the NCEER Workshop on Liquefaction Resistance of Soils (Youd and Idriss, 1997) and summarized in the "Guidelines for Liquefaction Hazard Assessment" (Dames & Moore) (appendix C). The paper by Robertson and Wride (1997), "Soil Liquefaction and its Evaluation Based on SPT and CPT", is the primary source of most guidelines for this procedure. The combination of CPT and SPT analyses used for this study will be further referred to as the "integrated liquefaction hazard evaluation procedure".

The results of the study indicated that liquefiable sediments lie beneath most bridge sites. Compared to the previous analysis, much less liquefiable material was identified at most bridge sites. At a few sites, however, some important layers were determined to be liquefiable that were missed in the previous study.

4.4.2 Methods of Study

4.4.2.1 Earthquake Design Parameters

The seismic study by Dames and Moore (1996) recommended a peak horizontal acceleration of 0.6 g and an earthquake magnitude of 7.0 for liquefaction hazard evaluations. These values were used for all analyses in this study and are consistent with predicted maximum values with 90 percent probability of not being exceeded in 250 years.

4.4.2.2 Integrated Liquefaction Hazard Evaluation Procedure

The integrated liquefaction hazard evaluation procedure combines both CPT and SPT liquefaction analysis methods. The combined analyses are more accurate in determining liquefiable sediments than previous methods because the two methods mutually compensate for individual weaknesses. Historically, the SPT based analysis method has been used for liquefaction hazard analyses. That procedure takes advantage of well-understood penetration data and provides soil samples for classification and laboratory analysis. The main weakness of the SPT method is the sporadic intervals at which SPT resistances are typically measured. This procedure provides a non-continuous soil profile requiring interpolation and extrapolation to create a continuous profile. The CPT procedure as developed by Robertson and Wride (1997) provides a nearly continuous soil record and liquefaction resistance interpretation, but lacks physical soil samples for soil classification and fines content determination. The absence of measured laboratory data to confirm the CPT analyses introduces uncertainty into the calculated results. The synthesized CPT and SPT procedure provides a continuous profile identifying potentially liquefiable sediments from the CPT log with sporadic confirmation and measured laboratory data from the SPT. Thus, the integrated liquefaction hazard analysis procedure provides more accurate and more certain results than from using each procedure individually. The integrated procedure was performed in the following order:

1) Potentially liquefiable and non-liquefiable materials were delineated using soil classification, water table depth, moisture content, and plasticity criteria.

2) The factor of safety (FS) against liquefaction was calculated for potentially liquefiable layers from the CPT logs and available laboratory test data.

3) An independent FS was calculated from SPT logs and available laboratory data. The FS from the SPT is used to confirm and compare results from the CPT analysis.

4) Potentially sensitive clays were identified based on CPT and laboratory data.

4.4.3 Sample Calculation of Cone Penetration Analysis Along I-15 Corridor

The following sample calculation illustrates the integrated liquefaction hazard evaluation procedure. The sample data are extracted from CPT, SPT and soil logs at the 48th South bridge site (appendix C).

4.4.3.1 Required Information

Earthquake design parameters:	
Peak horizontal acceleration (a_{max})	= 0.6 g
Earthquake magnitude (M)	= 7.0
Site data:	
Depth (z)	= 8.0 m
Water table depth (z_{wt})	= 3.0 m
Atmospheric pressure (Pa)	= 101 kPa
Unit weight water (γ_w)	= 9.81 kN/m^3
Average total unit weight soil (γ_{avg})	= 18.5 kN/m^3
Cone data: 45-RC-31	
Tip Resistance (q_c)	= 6390 kPa
Sleeve friction (f_s)	= 60 kPa
Thickness (H) for thin layer correction	= 3000 mm
SPT data: B-31	
Uncorrected blowcount (N)	= 11
Field calibrated energy ratio (ER)	= 58 %
Laboratory data:	
Fines Content (FC)	= 35 %
Liquid limit (LL)	= NOT MEASURED
Natural moisture content (w_n)	= NOT MEASURED
Clay content (CC)	= NOT MEASURED

4.4.3.2 Procedure, Equations, and Calculations

CPT Liquefaction Analysis

Step 1: Correct q_c for thin sand layer (q_{ck})

a) Identify thin sand layers using the process described in step 2, section 7.1, appendix B.

b) Calculate thin sand layer correction factor (K_c).

$$K_c = 0.5\ ((H/1000)-1.45)^2 + 1.0 \qquad \text{equation 7.1, appendix B}$$

where: H is in mm

$H < 1450$ mm

If $H > 1450$ then $K_c = 1.0$

$H = 3000$ mm; therefore,

$K_c = 1.0$

$q_{ck} = q_c * K_c$

$q_{ck} = 6390 \text{ kPa} * 1.0 = 6390 \text{ kPa}$

Step 2: Determine vertical effective stress (σ_{vo}')

$\sigma_{vo}' = (z * \gamma_{avg}) - (z - z_{wt}) * \gamma_w$
$\sigma_{vo}' = (8.0\ m * 18.5\ kN/m^3) - (8.0\ m - 3.0\ m) * 9.81\ kN/m^3 = 99.0\ kPa$

Step 3: Determine correction for overburden stress (C_Q)

$C_Q = (Pa/\sigma_{vo}')^{0.5}$ equation 5, Robertson and Wride (1997)
$C_Q = (101\ kPa / 99.0\ kPa)^{0.5} = 1.01$
note: C_Q is not to exceed 2.0 at shallow depths.

Step 4: Determine q_{c1N}, the tip resistance normalized for overburden pressure.

$q_{c1N} = (q_{ck} / Pa) * C_Q$ equation 5, Robertson and Wride (1997)
$q_{c1N} = (6390\ kPa / 101\ kPa) * 1.01 = 63.9$

Step 5: Determine the soil behavior index (I_c).

The soil behavior index is used to classify the soil into soil types. The higher the I_c, generally the finer grained and more plastic the soil. I_c is derived from a mathematical relationship between the normalized penetration resistance (Q) and the normalized friction ratio (F). I_c is used to estimate the fines content and the behavior type of the soil (Robertson and Wride, 1997). Refer to table 7.2, section 7.1, of appendix B for a listing of soil behavior type boundaries.

$I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$ equation 7.5, appendix B

where: $Q = (q_{ck} - \sigma_{vo}) / \sigma_{vo}'$
$F = [(f_s/(q_{ck} - \sigma_{vo})] * 100$
$\sigma_{vo} = z * \gamma_{avg}$
$\sigma_{vo} = 8.0\ m * 18.5\ kN/m^3 = 148\ kPa$
$Q = (6390\ kPa - 148\ kPa) / 99.0\ kPa = 63.1$
$F = [(60\ kPa / (6390\ kPa - 148\ kPa) * 100 = 0.96$
$I_c = [(3.47 - \log(63.1))^2 + (\log(0.96) + 1.22)^2]^{0.5} = 2.05$

note: according to table 7.2, section 7.1, appendix B, $I_c = 2.05$ classifies the soil as either a clean sand or a silty sand.

Step 6: Determine the equivalent clean sand normalized penetration resistance ($(q_{c1N})_{cs}$).

a) Fines content may be acquired in two ways: the preferred method is through laboratory gradation tests, and the less preferred method is through CPT based fines content approximation. The CPT fines content approximations are calculated with the following equation (Robertson and Wride, 1997):

$FC = 1.75 * I_c^3 - 3.7$
$FC = 1.75 * 2.05^3 - 3.7 = 11.6\ \%$

The laboratory measured FC = 35 % will be used in preference over the CPT fines content approximation.

b) Determine clean sand correction (Δq_{c1N}).

a) $\Delta q_{c1N} = 60$ if $FC \geq 35\ \%$
b) $\Delta q_{c1N} = 0$ if $FC \leq 5$ percent
c) $\Delta q_{c1N} = 2 * (FC - 5)$ if $5\ \% < FC < 35\ \%$

equation 6, Robertson and Wride (1997)

FC = 35 % satisfies equation a; therefore,
$\Delta q_{c1N} = 60$

c) Calculate $(q_{c1N})_{cs}$.

$(q_{c1N})_{cs} = q_{c1N} + \Delta q_{c1N}$
$(q_{c1N})_{cs} = 63.9 + 60 = 123.9$

Step 7: Determine the cyclic resistance ratio ($CRR_{7.5}$).

$CRR_{7.5} = 93 * ((q_{c1N})_{cs} / 1000)^3 + 0.08$ equation 7.7, appendix B
$CRR_{7.5} = 93 * (123.9 / 1000)^3 + 0.08 = 0.26$

where: $(q_{c1N})_{cs}$ is in the range of $30 < (q_{c1N})_{cs} < 160$

Step 8: Determine the stress reduction factor (r_d).

There are several different equations available to calculate r_d. The Dames and Moore Guidelines recommend determining r_d from the "average values" line of figure 20 at depths above 10 meters and carefully extrapolating the "average values" line for depths below 10 meters. Liao and Whitman developed a bilateral equation to approximate the "average values" line for all depths (1986). Their equation provides appropriate r_d values for average liquefaction conditions.

a) $r_d = 1.0 - 0.00765 * z$ $z \leq 9.2$ m
b) $r_d = 1.174 - 0.0267 * z$ $z > 9.2$ m
z = 8.0 m; therefore use equation a.
$r_d = 1.0 - 0.00765 * (8.0\ \text{m}) = 0.94$

Step 9: Determine the cyclic stress ratio (CSR).

$CSR = 0.65 * a_{max} * (\sigma_{vo} / \sigma_{vo}') * r_d$ equation 7.2, appendix B

$$CSR = 0.65 * 0.6 * (148\ kPa / 99.0\ kPa) * 0.94 = 0.55$$

note: The same CSR is used for both the CPT and SPT analyses.

Step 10: Determine the Magnitude Scaling Factor (MSF). For this analysis, Idriss' revised MSF (column 3, table 8) was used.

$$MSF = 173 * (M)^{-2.56}$$
$$MSF = 173 * (7.0)^{-2.56} = 1.19$$

Step 11: Determine the FS.

$$FS = (CRR_{7.5} / CSR) * MSF$$
$$FS = (0.26 / 0.55) * 1.19 = 0.56$$

SPT Liquefaction Analysis.

Step 1: Determine the correction factor based on the effective overburden pressure (C_n).

$$C_n = (Pa / \sigma_{vo}')^{0.5} \qquad \text{equation 7.10, appendix B}$$
$$C_n = (101\ kPa / 99.0\ kPa)^{0.5} = 1.01$$

Step 2: Determine the normalized blowcount $(N_1)_{60}$.

$$(N_1)_{60} = Cn * (ER / 60) * N \qquad \text{equation 7.9, appendix B}$$
$$(N_1)_{60} = 1.01 * (58.3 / 60) * 11 = 10.8$$

Step 3: Determine the equivalent clean sand normalized blowcount $((N_1)_{60cs})$.

a) Determine clean sand correction $(\Delta(N_1)_{60})$.

a) $\Delta(N_1)_{60} = 7$ for $FC \geq 35\ \%$
b) $\Delta(N_1)_{60} = 0$ for $FC \leq 5\ \%$
c) $\Delta(N_1)_{60} = (FC - 5) * 7 / 30$ for $5\ \% < FC < 35\ \%$

equation 3, Robertson and Wride (1997)

FC = 35 %; therefore,
$\Delta(N_1)_{60} = 7$

b) Calculate $(N_1)_{60cs}$.

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$
$$(N_1)_{60cs} = 10.8 + 7 = 17.8$$

Step 4: Determine the cyclic resistance ratio ($CRR_{7.5}$).

$CRR_{7.5}$ is taken from figure 7.9, appendix B.
For $(N_1)_{60cs} = 18$, $CRR_{7.5} = 0.19$

Step 5: Determine FS. The FS is calculated with the same equations found in steps 10 and 11 of the CPT analysis.

$$FS = (CRR_{7.5} / CSR) * MSF$$
$$FS = (0.19 / 0.55) * 1.19 = 0.42$$

The FS determined by both the CPT and SPT based analyses are seldom equivalent. For this layer at the 48th South Bridge site the CPT based FS is about 20 percent greater than the SPT based FS. This discrepancy results from inconsistency of soil layering typical of alluvial deposits and from differences in fines content corrections. Currently, the procedures used to correct penetration resistance for fines content are being reviewed to refine accuracy and to better correlate CPT and SPT based analyses.

Soil Classification, Moisture Content, Penetration and Plasticity Criteria

This procedure applies four limiting physical factors which classify qualifying sediments as non-liquefiable: maximum penetration resistance, water table depth, soil classification and plasticity. None of these factors are accounted for in the CPT or SPT factor of safety analyses; therefore, they were applied afterward to further identify non-liquefiable sediments.

Limit 1: Maximum penetration resistance.

If $(q_{c1N})_{cs} > 160$ the material is classed as non-liquefiable (Robertson and Wride, 1997)
If $(N_1)_{60cs} > 30$ the material is classed as non-liquefiable (Seed and Idriss, 1985).

Limit 2: Soil lying above the water table. If the water table at a site is deeper than 15 m, the site is classed as non-liquefiable (Youd, 1998).

If $z < z_{wt}$ the material is non-liquefiable.

Limit 3: Standard criterion. A fine-grained soil must satisfy <u>all</u> the following constraints to be classed as liquefiable (Seed and Idriss, 1985):

Percent finer than 0.005 mm < 15 %
$LL < 35\%$
$w_n > 0.9\ LL$

Limit 4: Clay classification by CPT approximation.

The soil behavior index, I_c, calculated from CPT data was developed by Robertson and Wride (1997) to define boundaries between various soil types including a boundary between plastic and non-plastic material. This latter boundary also distinguishes between liquefiable and non-liquefiable soils. Figure 23 shows a recommended "sandy soil-clayey soil" boundary at I_c = 2.60 (Robertson and Wride, 1997). Based on this boundary, a soil with $I_c > 2.60$ would be classed as non-liquefiable, whereas a soil with $I_c \leq 2.60$ would be classed as liquefiable.

The I_c boundary is critical. Typically, saturated normally consolidated fine grain soils have low tip resistances that contribute to low calculated factors of safety. A normally consolidated soil with $I_c = 2.60$ will be classed as sandy silt or silty sand with a low factor of safety against liquefaction and a soft soil with $I_c = 2.61$ will be classed as a non-liquefiable clayey silt or clayey sand. To better define this boundary, recommended practice is to carefully compare soil samples with classifications estimated from the CPT data. Local correlations can then be developed to improve classification from the CPT test. The local calibration process is further described in section 4.4.4.3.

Check for Potentially Sensitive Soils.

Fine grained soils meeting the following criteria may be extra sensitive and have potential to catastrophically lose strength during earthquake shaking (Youd, 1998):

Sensitivity > 4
Liquidity Index > 0.6
$w_n > 0.9$ LL
$q_{c1N} <$ 1Mpa or $(N_1)_{60} < 5$
UCS classifications of CL or ML or
AASHTO classifications A-4, A-2-4, A-6, or A-2-6

4.4.3.3 Spreadsheet Application

The entire integrated liquefaction analysis procedure described in section 4.4.3 was programmed into a spreadsheet. Factors of safety based on CPT and SPT are both calculated for comparison. The CPT analysis was programmed to apply measured fines content data as a preference to CPT based approximations. All of the criteria that identify non-liquefiable soils such as penetration, soil classification, moisture content, I_c or plasticity values are also applied in the spreadsheet analyses.

4.4.4 Liquefaction Resistance at Bridge Sites

4.4.4.1 Quality of Available Data

State-of-the-art CPT equipment with calibrated sensors and standard SPT equipment with field calibrated energy ratios were used. The laboratory data included gradations, Atterberg limits, and natural moisture contents. Water table depths and surface elevations were measured for each CPT and SPT location. The CPT penetration data was averaged over approximately 0.3 m intervals to simplify the analyses.

4.4.4.2 Availability of Data

Each bridge site evaluated had recent SPT data but sparse amounts of laboratory data. In addition, all bridge sites had CPT data with the following exceptions: I-215 Interchange, 7200 South, Center Street in Midvale, and Wasatch street. Even where borings were located near CPT soundings soil samples were usually only taken at intervals of a meter or greater leaving large parts of the cone log unverified.

4.4.4.3 Soil Type as Determined by the Soil Behavior Type Index, I_c

Critical Nature of the I_c Value

In the absence of grain size data from a parallel confirmatory borehole, Ic (equation 7.5 appendix B) is used to estimate fines content and to identify soil behavior type. As suggested in section 4.4.3.2, (limit 4) a local verification of I_c can more accurately isolate the boundary between clayey soils and sandy soils.

Figure 23 divides the I_c values into seven zones of soil type. I_c values in zones 2- 4 are classified as non-liquefiable due to clay content and high liquid limits. Zones 6 and 7 contain

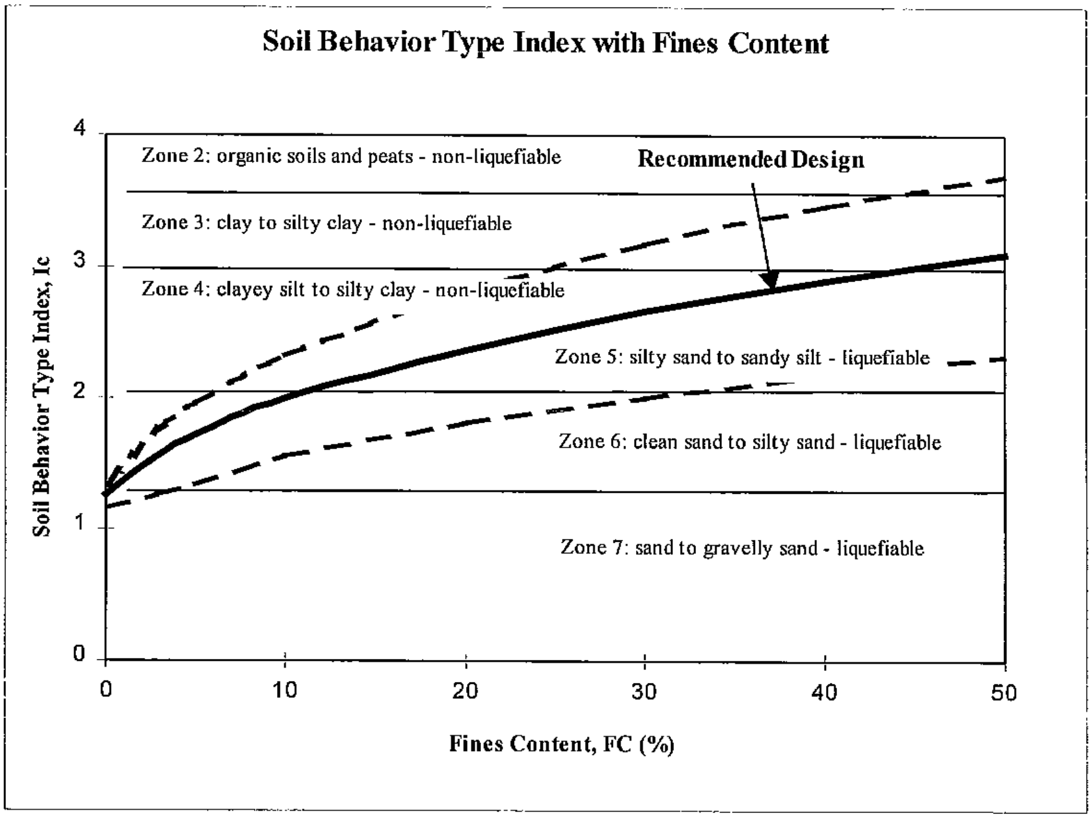


Figure 23: Variation of soil behavior type index (I_c) with fines content (Robertson and Wride, 1997).

clean sands and gravels that could be liquefiable depending on penetration resistance. Zone 5, I_c range of 2.05 - 2.60 (figure 23), separates the clayey soils from the clean granular soils. The sediments in this zone are silty sands to sandy silts which are difficult to classify in terms of liquefiability; potentially high silt contents, high plastic limits, and significant amounts of undetected clay could classify many sediments in this zone as non-liquefiable. For this preliminary study, zone 5 soils with factors of safety less that 1.0 were classified as liquefiable for conservatism; however, as the I_c increases, the certainty that the layer is liquefiable decreases. The Salt Lake County section of the I-15 Corridor is underlain by many sediments which fall into zone 5. To determine what percentage of possibly liquefiable sediments fall into this questionable range of I_c values, a survey of I_c values for each liquefiable layer was performed.

Local Calibration of I_c

A local calibration was developed to clarify the boundary value between zones 4 and 5. Approximately 650 I_c values between 2.41 and 2.85 along with soil classifications from parallel soil samples were compared. In some cases the compared CPT sounding and SPT borehole were between 50 m and 150 m apart, thus correlating the I_c with distant soil samples introduced some uncertainty. The data was widely scattered but did produce a conservative boundary at $I_c = 2.65$. This boundary correctly predicted clayey soils with an 80 percent accuracy or higher (see figure 24). I_c values between 2.41 and 2.65 correctly predicted clayey soils with 40 percent to 70

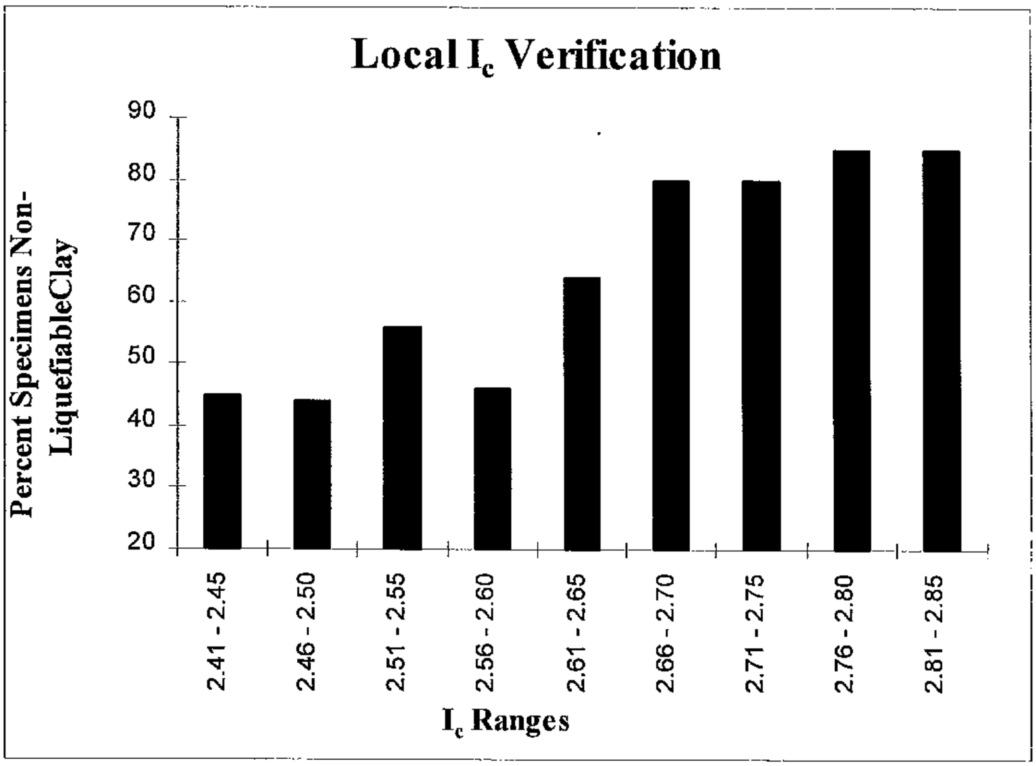


Figure 24: Local calibration of I_c value from CPT versus prediction of non liquefiability (LL > 35 or w_n < 0.9 LL) from laboratory tests of fine-grained specimens.

percent accuracy. The more conservative boundary of 2.65 was selected for this study. Use of this conservative boundary means that many more non-liquefiable soils will be classed as liquefiable than liquefiable soils classed as non-liquefiable based solely on CPT analyses. For this preliminary analysis, this conservatism is appropriate. Thus, more detailed site investigations and laboratory investigations are likely to reduce the calculated hazard compared to this preliminary study.

Survey of I_c Values for Liquefiable layers

The I_c values of all liquefiable layers were analyzed to determine the distribution of soil type throughout the liquefiable layers. The distribution, figure 25, shows most of the liquefiable material to be in the higher I_c range of zone 5 and that relatively few layers classify into zone 6. Eighty-four percent of all possibly liquefiable sediments are classed as zone 5 silty sands to sandy silts. The remaining 16 percent are classified as zone 6 - 7 clean sands. The local I_c calibration determined that soils with I_c values ranging from 2.41 to 2.65 had a 60 percent likelihood of being granular and a 40 percent likelihood of being clay rich and non-liquefiable. As the I_c decreases, the percentage of soils likely to be miss-classified as non-liquefiable due to

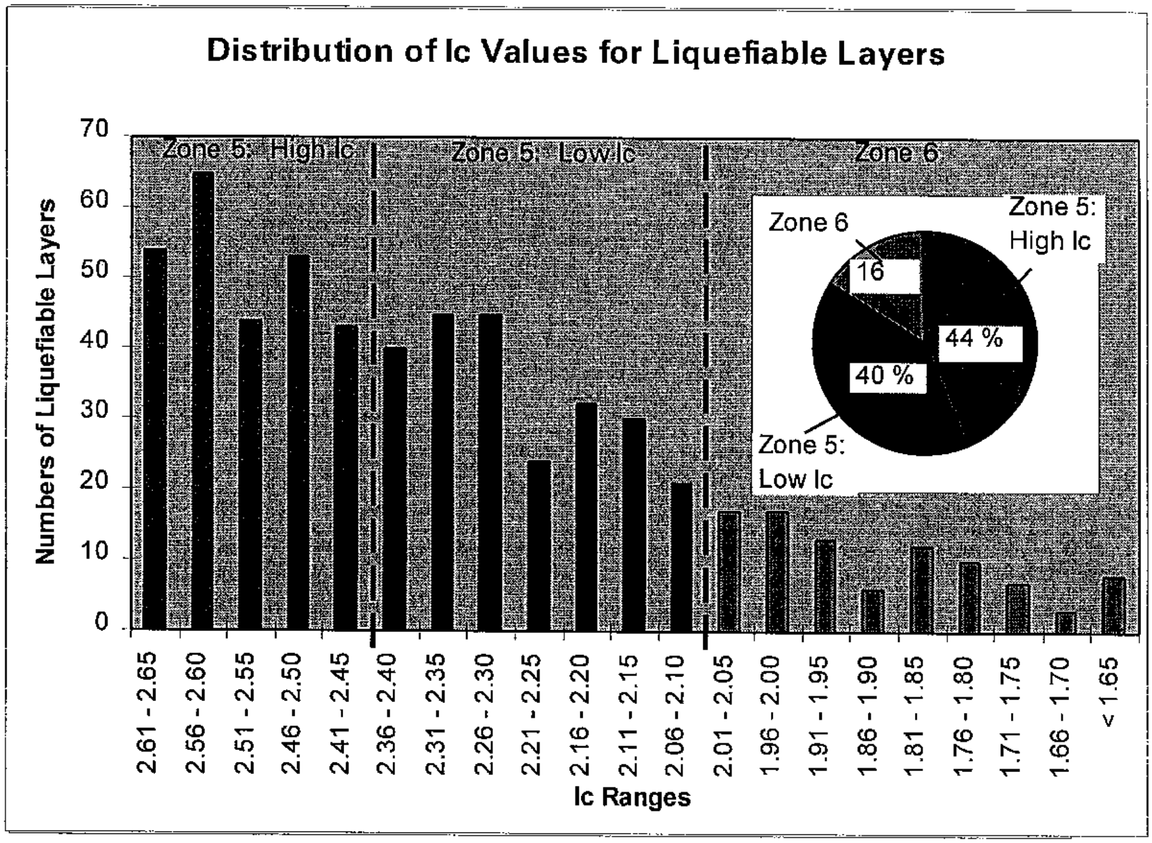


Figure 25: Distribution of I_c values for liquefiable layers only. Approximately 85 % of liquefiable layers are classified into zone 5.

clay content or plasticity also decreases. The certainty that a sediment is liquefiable is higher for lower I_c values. Forty-four percent of the layers determined to be liquefiable are in the I_c range with the least certainty of liquefiability.

Figure 26 shows three liquefaction logs with liquefiable material divided into I_c ranges. The 48th South bridge site has a significant amount of liquefiable material. The liquefiable sediment is primarily zone 6 clean sands and has the highest likelihood of being correctly predicted. Liquefiable layers at the 5th South Viaduct consist of relatively equal amounts of low I_c zone 5 and zone 6 materials. Thus, the liquefiable soils at this site have a moderate to high certainty of being correctly predicted. The analysis at the I-80 Interchange indicates that approximately 6 meters of liquefiable sediments lie in the upper 10 meters; however, this thick liquefiable layer has I_c values ranging from 2.4 to 2.6 which have a low to moderate certainty of being correctly predicted. Overall, 85 percent of liquefiable layers beneath all bridge sites have a low to moderate certainty of being correctly predicted. More detailed laboratory analyses and site investigations are likely to reduce the amount of calculated liquefiable sediments under most of these bridge sites.

4.4.5 Results

The results of this study are presented graphically in several figures along with supporting discussion. The graphical results are in three forms: (1) graphs of summary liquefaction logs (figures 45 - 48), (2) copies of spreadsheets listing results of the integrated procedure for each bridge site (appendix C), and (3) tables summarizing the results of liquefaction analyses for individual bridge sites (appendix C). These results are found in section 6.10.

4.4.6 Chapter Conclusions

The analyses and results contained in this study yield the following conclusions:

1. Some potentially liquefiable sediments lie beneath nearly all bridge sites in the I-15 Corridor. Only the 72nd South and Center Street in Midvale bridge sites appear to be free of liquefiable material. Most sites are underlain by cumulative thicknesses of susceptible sediments ranging from 1 m to 7 m in the upper 15 m. Liquefaction of these sediments could adversely affect embankment stability, lead to lateral spread, or generate ground settlement. Also, many sites contain liquefiable material below 15 m. Liquefaction of these layers could adversely affect the load capacity of some deep foundations.

2. The integrated liquefaction hazard analysis procedure used for this study has some important advantages. This procedure produces more certain boundaries between layers by utilizing the continuous soil profile and analysis methods of the CPT that were verified at intermittent intervals by analyses utilizing SPT and laboratory data.

3. By using the integrated procedure with recent and complete soil data, the total amount of liquefiable material was reduced by 54 percent compared to the study using older, less quality data. Also, some possibly liquefiable layers were added by this analysis.

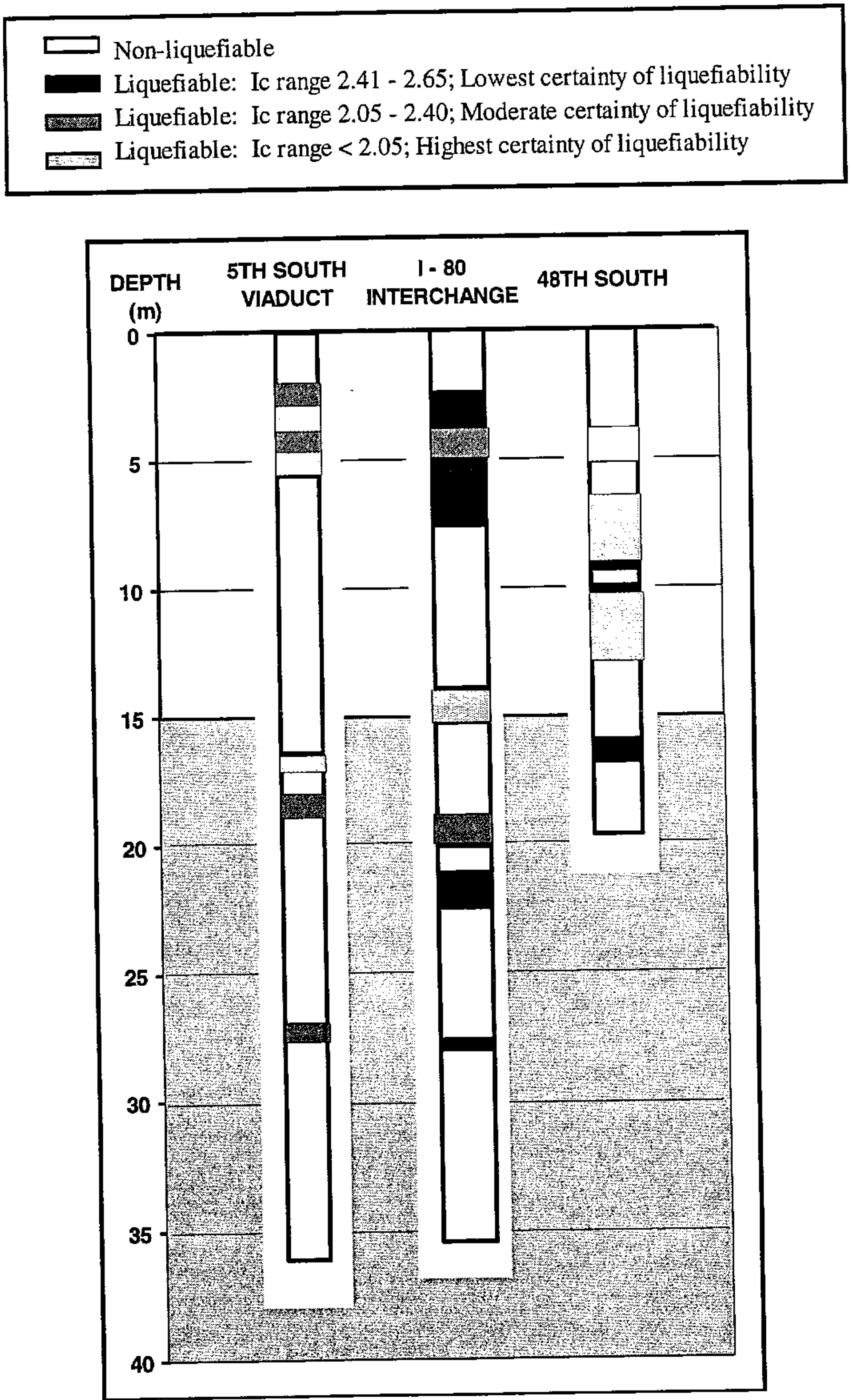


Figure 26: Example bridge sites with liquefiable layers separated into Ic ranges.

CHAPTER 5 - LIQUEFACTION-INDUCED GROUND FAILURE

Liquefaction alone is not the main cause of bridge damage. Structural damage occurs due to liquefaction-induced loss of embankment stability and deformation, lateral spread, excessive vertical settlement, or loss of foundation bearing capacity. If the analysis of standard penetration or cone penetration resistances result in confirmed susceptibility of soil layers to liquefaction, with a factor of safety less than 1.2, these adverse consequences should be analyzed. This section describes how each of these hazards is evaluated and provides examples of how each step is implemented. The geotechnical data available to this study for bridge sites in the I-15 corridor, however, were generally inadequate for calculation of meaningful estimates of ground displacements. Thus, only a few estimates were made to illustrate the procedure.

5.1 Embankment Stability Analysis

As subsurface deposits of soil liquefy, temporary strength loss occurs creating an ideal condition for possible slope failure. For the interstate bridges, embankments constructed as approach fills and grade separations are the most likely sites for slope failure. The slope stability can be analyzed using a standard limit equilibrium analysis program such as UTEXAS2.

The embankment should be analyzed using residual strengths in the liquefiable layers. If the factor of safety is less than 1 (FS < 1), the site could become unstable if subsurface layers liquefy. This instability could cause significant slope deformation during an earthquake. Such instability at bridge sites could lead to large ground movements that could damage foundations and the superstructure. Additional analyses are needed at such sites to develop mitigative measures to reduce or eliminate the hazard.

If FS > 1, the embankment should be analyzed to determine the pseudostatic horizontal acceleration, a_h, that reduces the factor of safety to unity, FS = 1. This should be performed utilizing the residual strengths in the liquefiable layers. If this pseudostatic acceleration is determined to be greater than the peak horizontal acceleration caused by the earthquake, $a_h \geq a_{max}$, then the site would be stable under the horizontal accelerations produced by the earthquake shaking. If $a_h < a_{max}$, deformation could occur and thus should be calculated to determine if mitigation is necessary.

<u>Example of Embankment Stability Analysis</u>

The embankment used for this example is the exit at the 600 South southbound off ramp. The embankment is approximately 9 m high. The soil information was gathered from boreholes DH-15 and DH-15A, which are approximately 50 m from the ramp fill. Gerber (1995) compiled extensive information for this site including strengths for the layers of sands and clays as shown in figure 22. The simplified procedure, using data from the upper 25 m of this site, indicated a primary liquefiable layer at a depth from 1.2 m to 4.0 m and a thinner layer at 17.1 m to 17.7 m. The layers deeper than 15 m were ignored for this stage of the analysis because they were too deep to affect slope stability. The stability analysis was performed using UTEXAS2 which implemented Spencer's method for total equilibrium. The objective of the stability analysis is to determine if the embankment is statically stable assuming liquefaction occurs.

The residual strengths for the liquefied layers between 1.2 m and 2.4 m were found using the procedure developed by H. Bolton Seed as presented by Seed and Harder (1990). The $(N_1)_{60}$ values calculated for these layers were also corrected for fines content, $(N_1)_{60\text{-}cs}$. These values were then used with figure 27 to determine the average residual undrained shear strength of the liquefied layer. Using this strength, UTEXAS2 calculated a factor of safety (FS) of 1.46 with the resulting failure plane shown in figure 28. Even though the embankment was shown to be stable under liquefied conditions, FS > 1, the potential for deformation still exists. The pseudostatic acceleration, a_h, was calculated to determine if a deformation analysis was necessary. Using a residual strength in the liquefiable layer, this calculation resulted in an acceleration, a_h, of 0.1 g to cause FS = 1. Because the $a_h < a_{max}$ (a_{max} = 0.5 g at 600 South), the deformation potential of the embankment was calculated to determine if it would damage the structure.

5.2 Embankment Deformation Analysis

Sites with expected horizontal accelerations greater than the pseudostatic acceleration, should be evaluated for deformation potential (D). D can be calculated using a Makdisi and Seed, mechanistic (Newmark) or finite element analysis. If deformations are found to be tolerable for the bridge structure, the embankment can be considered stable and the screening guide is followed to the lateral spread analysis. Tolerable deformations vary from structure to structure, but could be as high as 100 mm to 200 mm for a well-built new structure.

Example of Embankment Deformation Analysis

The deformation potential of the 600 South off-ramp was calculated using the finite element analysis program QUAD4M. The program calculated an acceleration time-history of the embankment. All accelerations that exceeded the pseudostatic acceleration were double integrated with respect to time to give the deformation. The total deformation was calculated to be less than 40 mm (< 1.6 in) which would be acceptable for the 600 South structure.

5.3 Lateral Spread Analysis

Youd (1993b) states that soil liquefaction may lead to flow failures, lateral spreading, or ground oscillation. He states that flow failures usually occur on steep slopes (greater than 6 percent) and are associated with large movements and substantial disruption of the soil mass. Ground oscillation generally occurs on flat ground (less than 0.1 percent) and is usually not associated with large permanent displacements. Lateral spreading is located somewhere between these two ground failures combining components of each (Youd, 1993b). Lateral spread has been the most common cause of liquefaction-induced bridge damage. Because slopes at I-15 bridge sites are too gentle for flow failure to occur, lateral spread is the most damaging

Recommended Fines Correction for S_r Evaluation Using SPT Data

Percent Fines	N_{corr} (blows/ft)
10%	1
25%	2
50%	4
75%	5

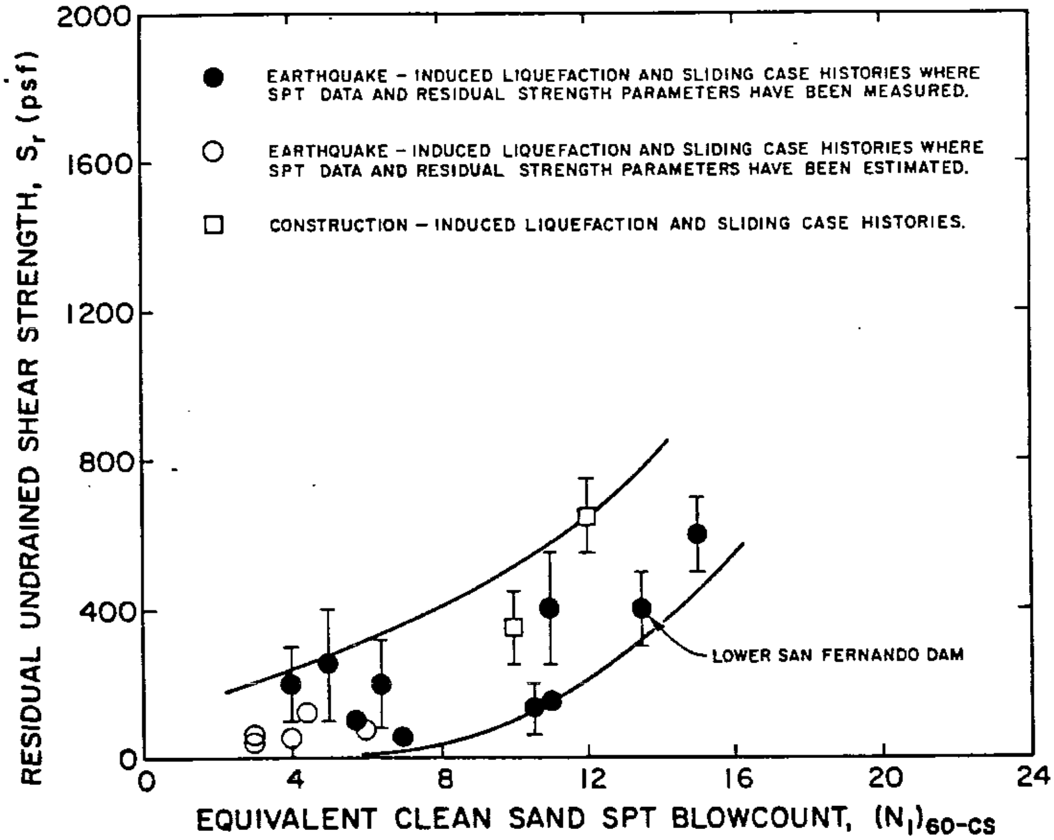


Figure 27: Relationship between residual strength and corrected SPT resistance (after Seed and Harder, 1990)

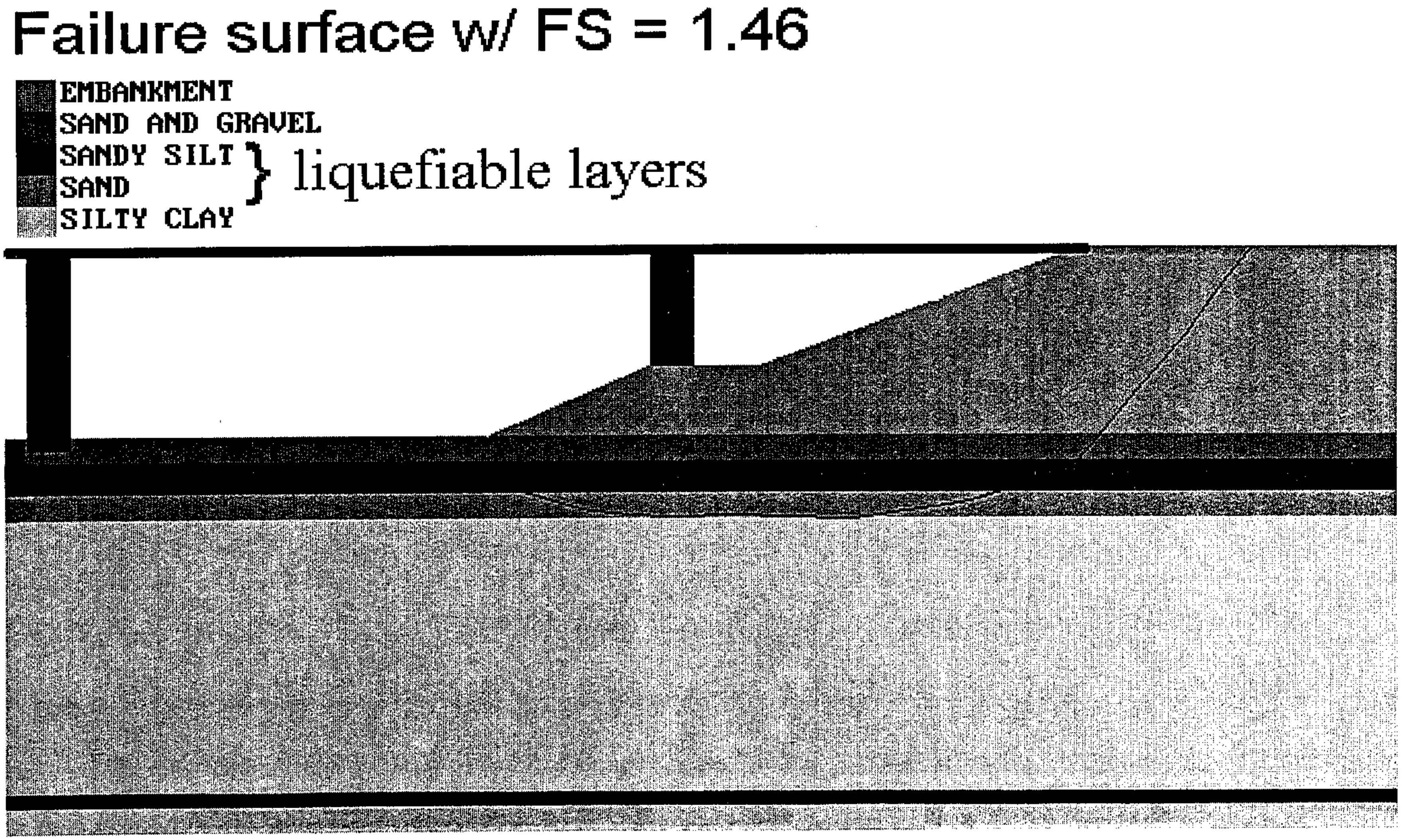


Figure 28: Profile of 600 South bridge site showing critical failure plane with factor of safety, FS, of 1.46

consequence of liquefaction. Youd (1998) uses empirical equations developed by Bartlett and Youd (1995) to calculate lateral spread displacement. These equations are:

for free face conditions:

$$\text{LOG } D_H = -16.3658 + 1.1782\text{ M} - 0.9275\text{ LOG R} - 0.0133\text{ R} + 0.6572\text{ LOG W} + 0.3483\text{ LOG } T_{15} + 4.5270\text{ LOG } (100 - F_{15}) - 0.9224\text{ D50}_{15} \tag{6a}$$

for ground slope conditions:

$$\text{LOG } D_H = -15.7870 + 1.1782\text{ M} - 0.9275\text{ LOG R} - 0.0133\text{ R} + 0.4293\text{ LOG S} + 0.3483\text{ LOG } T_{15} + 4.5270\text{ LOG } (100 - F_{15}) - 0.9224\text{ D50}_{15} \tag{6b}$$

where: D_H = estimated lateral ground displacement in meters
M = earthquake magnitude (moment magnitude)
R = horizontal distance from the site to seismic energy source, in kilometers
T_{15} = cumulative thickness of saturated granular layers with corrected blow counts less than 15, $(N1)_{60} < 15$, in meters
F_{15} = average fines content (fraction of soil sample passing a No. 200 sieve) for granular layers included in T_{15}, in percent
$D50_{15}$ = average mean grain size in granular layers included in T_{15}, in millimeters
S = ground slope, in percent
W = ratio of the height of the free face (H) to the horizontal distance to the base of the free face (L), in percent

If the horizontal displacement due to lateral spreading (D_H) is tolerable (typically $D_H < 200$ mm for new, well-built structures) then minimal lateral spread hazard exists and the site may be classified as not susceptible to lateral spread and the screening continues to the next step. If $D_H \geq 100$ mm then a significant lateral spread hazard exists and a detailed site study and possible remedial measures are required.

Four of the variables in the above equations, F_{15}, $D50_{15}$, S, and W, must usually be estimated. F_{15} and $D50_{15}$ can be approximated from soil descriptions and S and W are estimated using topographic maps of the site. If these variables can not be reliably determined, the calculated lateral displacements will not give meaningful results. Due to the dearth of reliable data, lateral spread hazard was not calculated for bridge sites in Utah except for the following illustrative example.

Example of Lateral Spread Evaluation

The 600 South off-ramp does not have any free-face conditions so the ground-slope equation (6b) was used. The only layer that poses a significant lateral spread hazard is the upper liquefiable layer at a depth of 1.2 m to 2.7 m. The lower liquefiable layers are too thin and deep to pose a lateral spread hazard (depth > 15 m). The following measured data and best estimates for site material properties were used in the analysis. Many of these values are outside the limits specified by Bartlett and Youd (1995).

$M = 7.0$
$R = 3.0$ km (under 5.0 km limit)
$S = 0.25$ %
$T_{15} = 1.5$ m
$F_{15} = 75$ % (over 50 % limit)
$D50_{15} = 0.06$ mm (under 0.1 mm limit)

Entering these values into equation (6b) yields:

$$\text{LOG } DH_1 = -15.7870 + 1.1782(7.0) - 0.9275 \text{ LOG}(3) - 0.0133(3) + 0.4293 \text{ LOG}(0.25) + 0.3483 \text{ LOG}(1.5) + 4.5270 \text{ LOG}(100 - 75) - 0.9224(0.06) = -1.946$$

$DH_1 = 11$ mm

If the site only experienced 11 mm of displacement, the bridge would likely be undamaged. However, because several variables are outside the specified limits, the result is uncertain. The analysis was conducted a second time with the values adjusted within the nearest limiting value listed in table 11.

$M = 7.0$
$R = 5$ km (limiting value)
$S = 0.25$ %
$T_{15} = 1.5$ m
$F_{15} = 50$ % (limiting value)
$D50_{15} = 0.1$ mm (limiting value)

Entering these values into equation (6b) yields:

$$\text{LOG } DH_2 = -15.7870 + 1.1782(7.0) - 0.9275 \text{ LOG}(5) - 0.0133(5) + 0.4293 \text{ LOG}(0.25) + 0.3483 \text{ LOG}(1.5) + 4.5270 \text{ LOG}(100 - 50) - 0.9224(0.1) = -0.8525$$

$DH_2 = 140$ mm

Note that the actual displacement is likely between 11 mm and 140 mm. The displacement using the actual measured data is uncertain because it does not fall into the limits used to derive the lateral spread equations. The displacement using the limiting values produces high but uncertain results because the values were adjusted to the nearest applicable limit and do not represent actual conditions at the site. Even though 140 mm is a high estimate, a strong new bridge should be able to withstand 140 mm of movement with minimal damage. The potential displacements at this site are small enough not to require additional analysis. A weak older structure, however, might be damaged and would need additional investigation.

Table 11: Ranges of input values for independent variables for which predicted results are verified by case-history observations (after Bartlett and Youd, 1995)

Input Factor	Range of Values in Case History Database
Magnitude	$6.0 < M < 8.0$
Free-face Ratio	$1.0\% < W < 20\%$
Ground Slope	$0.1\% < S < 6\%$
Thickness of Loose Layer	$0.3\ m < T_{15} < 12\ m$
Fines Content	$0\% < F_{15} < 50\%$
Mean Grain Size	$0.1\ mm < D50_{15} < 1\ mm$
Depth to Bottom of Section	Depth to Bottom of Liquefied Zone , 15 m

5.4 Ground Settlement Analysis

Settlement occurs because cohesionless materials densify when subjected to repeated cyclic shear strains, like those created by earthquake shaking. The principal method for calculating settlement is a procedure developed by Tokimatsu and Seed (1987). The basic idea behind their method is that earthquakes cause volumetric strain due to cyclic shear strains which compact the soils, or due to reconsolidation of granular materials after the earthquake-induced pore pressures dissipate. They have shown that the volumetric strain is mainly a function of the relative density of the soil and the cyclic stress ratio (CSR) corrected to a magnitude 7.5 earthquake. Because relative density can be estimated from corrected standard penetration resistance, $(N_1)_{60}$, Tokimatsu and Seed established a relationship between the CSR, $(N_1)_{60}$, and the volumetric strain. This relationship, shown in figure 29, is a compilation of data gathered from laboratory testing and field case histories. Based on the CSR and $(N_1)_{60}$ calculated in the penetration analysis, a volumetric strain for clean sand may be determined from the figure.

Because figure 29 gives results for clean sands only, blow counts should be corrected for fines content. Youd (1998) states that a conservative approach to calculating volumetric strain is to assume a clean sand for all soils because clean sand results in larger calculated settlements than a silty soil. If a settlement corrected for fines is desired, Bartlett (1995) uses the $(N_1)_{60}$-CRR plot in figure 21 to correct the $(N_1)_{60}$ for fines before use in figure 29 This correction is made by extending a line from the $(N_1)_{60}$ value on the abscissa vertically to the appropriate fines content, horizontally from there over to the 5 percent fines content line and then vertically back to the abscissa for the clean-sand corrected $(N_1)_{60}$. The 35 percent fines-content line can be used for fines contents greater than 35 percent since it would be a conservative estimate of settlement. This fines-corrected $(N_1)_{60}$ is then used to find the volumetric strain from figure 29.

Once the volumetric strain has been determined for a layer, the settlement for that layer is calculated by multiplying the strain by the layer thickness associated with the particular $(N_1)_{60}$.

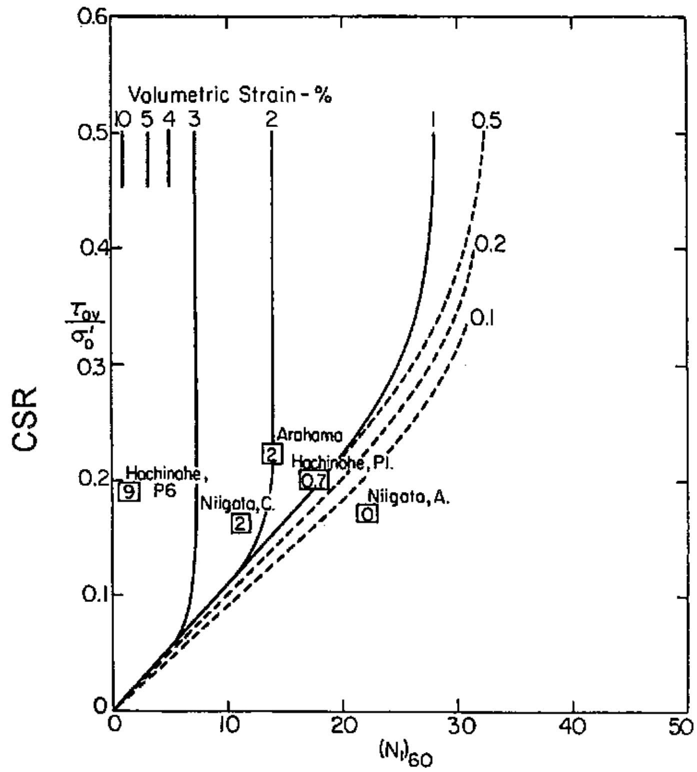


Figure 29: Chart for determination of volumetric strain using CSR and $(N_1)_{60}$ (after Tokimatsu and Seed, 1987)

The total settlement is merely the sum of each individual layer's settlement. New, well-built structures on deep foundations should be safe against settlements as great as 100 mm. Small settlements as low as 25 mm could pose a significant settlement hazard to older brittle structures. If settlements are sufficient to cause bridge damage, such as settlement of shallow footings or detrimental down-drag forces on deep foundations, then a more detailed site study should be initiated to define the settlement hazard and to design possible remedial measures.

Example of Settlement Analysis

The calculation of settlement was programmed into the same database that was used to calculate the liquefaction potential. Settlement was therefore easily determined for most of the sites for which site-specific data were analyzed. It should be noted that even though liquefaction was calculated to depths greater than 15 m, the settlements shown in appendix A are only to depths of 15 m. As stated in the penetration analysis, layers at depths greater than 15 m are probably too old and cemented to further densify under seismic shaking and would not produce significant settlement.

The settlement results shown in appendix A were calculated for bridge sites with reliable blow counts (those taken with standard equipment). The sites with the pound sign (#) have a lower confidence because the blow counts taken with non-standard equipment were converted to standard penetration (N_m) using an equation or graphical correlation.

To date (1998), no settlement equations or correlations have been developed for use directly with CPT data. To use CPT data to calculate settlement, the cone data must be converted to SPT data and used with figure 29. This would introduce some error into the calculation especially if corrections for fine contents are required. Section 4.4 by Gilstrap explains the problems associated with the estimation of fines contents and liquefaction using the soil behavior index (I_c). Due to the potential for variation in interpretation of cone results and the need for correction to SPT, the CPT data for the I-15 corridor was not used in settlement calculation.

The 600 South area provides a good example of a settlement calculation. Settlement was calculated for the cohesionless layers at depths of 1.2 m to 2.7 m and 2.7 m to 4.0 m. The uncorrected values of $(N_1)_{60}$ and the CSR were taken from the penetration analysis calculated in section 4.3. Table 12 shows the fines corrections, estimated volumetric strains, and settlements. Although the settlement, D_V, is greater than 25 mm (for an older existing structure), a well-constructed new bridge should not be affected by this much settlement. Thus, this site was classified as low hazard for excessive ground settlement.

Table 12: Values used in settlement calculations for 600 South off-ramp. Layers are at depths of 1.2 m and 2.7 m

LAYER THICKNESS	$(N_1)_{60}$	$(N_1)_{60\text{-}cor}$	CSR	VOLUMETRIC STRAIN (%)	SETTLEMENT
1.5 m	11	19	0.476	1.5	23 mm
1.2 m	19	28	0.524	0.9	11 mm
			TOTAL SETTLEMENT:		34 mm

5.5 Bearing Capacity Analysis

A bearing capacity analysis which considers liquefaction of subsurface layers is needed to assure that shallow foundations or piles do not settle, buckle, or penetrate into liquefied layers, causing potentially large differential deformations within the bridge structure. A bearing capacity hazard exists if the bridge foundation relies heavily on potentially liquefiable layers for shallow foundation bearing strength or frictional or end bearing strength for piles. Youd (1998) states that if a bridge has a deep foundation in which piles penetrate the liquefiable layers and transfer the bridge loads to deeper layers, the bridge should not suffer any structural damage from liquefaction as long as the lateral spread and settlement displacements are acceptable.

The analysis consists of comparing the liquefiable layers identified in the standard or cone penetration steps (sections 4.3 or 4.4) with the bearing strata as shown on foundation plans. If the piles have adequate capacity without relying on the liquefiable layers, or if calculated lateral or vertical displacements are tolerable, the bridge may be classified as low liquefaction hazard and low priority for further study.

Example of Bearing Capacity Analysis

600 South has liquefiable layers between the depths of 1.2 m to 4.0 m, 17.1 m to 17.7 m, 24.4 m to 25.6 m, 26.5 m to 27.7 m, and 28.6 m to 31.1 m. Foundation plans were not found for the existing structure so an analysis of bearing capacity could not be made at this time. However, a bearing capacity analysis should be made to assure that liquefaction will not adversely affect the safety of the bridge. If the piles or caissons were to terminate in these layers or if the layers were used as principal bearing strata, foundation settlement might occur. For any future structure at the site, care should be taken not to rely on the liquefiable layers for significant bearing strength.

5.6 Summary of Analyses of Liquefaction-Induced Ground Failure

The simplified procedure indicated that liquefiable layers lie beneath the existing 600 South off-ramp at depths of 1.2 m and 2.7 m and several thinner layers below 15 m. The first consequence of liquefaction considered was embankment stability. Standard slope stability analyses indicate that the 9 m embankments at the site are stable under dynamic conditions with a factor of safety of 1.46. In a second analysis, estimated embankment deformations were small with predicted movements of less than 40 mm due to seismic shaking. The third consequence of liquefaction was ground displacement due to lateral spread. Application of MLR equations indicate displacements of less than 150 mm, which should be non-damaging to well constructed bridges. The fourth consequence was ground settlement. Simplified analyses indicate settlements less than 35 mm, which are generally non-damaging. The final consequence was bridge foundation stability. By assuring that structural loads are transferred downward to competent layers, liquefaction can be accommodated at this site without adverse consequences to the highway system.

These example analyses indicate that liquefaction hazard can be mitigated at the 600 South off-ramp through structural measures. If predicted embankment deformations or lateral spread displacements had been potentially damaging, other mitigative measures, such as soil improvement, may have been necessary.

CHAPTER 6 - RESULTS BY HIGHWAY SEGMENT

The various steps outlined in chapters 3-5 are used to determine liquefaction susceptibility of bridge sites and prioritize them for future study. Youd (1998) lists the prioritizations as:

Priority I sites: Bridge sites with the highest priority for further investigation and possible mitigation were categorized as Priority I sites. These sites are likely to be underlain by liquefiable sediment or sensitive clay that could induce damaging ground or foundation displacements during an earthquake. Liquefiable sediment at these sites was either confirmed or the available information was insufficient to eliminate the possibility of liquefiable sediment. A second criterion is that the site is located in an area highly vulnerable to ground failure, such as river crossings, near lakes or other bodies of water, near a steep slope, or approached by thick embankments (greater than 5 m) overlying soft sediment.

Priority II sites: Bridge sites with the second highest priority for investigation and possible mitigation are localities confirmed to be underlain by liquefiable sediments or sensitive clay, but where the available site information is insufficient to fully evaluate ground or foundation stability hazards. Priority II sites are located away from rivers, other bodies of water, steep slopes, or thick embankments overlying soft sediment (otherwise they would be classed as Priority I sites). Liquefaction at these sites could cause ground settlement and possible foundation instability, but damaging lateral ground displacements are unlikely. These sites are assigned a moderate priority for further investigation.

Priority III sites: Bridge sites with third priority for further investigation are those with insufficient available geotechnical information to fully assess the liquefaction hazard. These sites are also located away from rivers, other bodies of water, steep slopes, or thick embankments overlying soft sediment (otherwise they would be classed as Priority I sites). Liquefaction at these sites could cause ground settlement and possible foundation instability, but damaging lateral ground displacements are unlikely. These sites are assigned a moderate to low priority for further investigation.

Priority IV: Bridge sites with the lowest priority for further investigation are those where the screening evaluation indicated very low liquefaction susceptibility or ground failure hazards. These bridges were assigned a low priority for further investigation or mitigation.

River crossings have the highest priority for further study because they have the highest potential for lateral spread. The primary results from this study are a tabulation of bridge sites with various priorities for future investigation and possible remediation. For bridges characterized by high or very high priorities, depths and thicknesses of potentially liquefiable layers beneath these sites are plotted on graphs for various highway segments throughout the state of Utah as assessed using the simplified procedure. This chapter is organized by highway segment (mainly by county) starting at the north and progressing south.

6.1 Values Used in Analyses

The peak accelerations used in the simplified procedure were estimated using figure 7 which shows contours of peak horizontal acceleration with a 10 percent probability of being exceeded in 250 years. The earthquake magnitudes were taken from figure 5 which shows the seismic source zones and earthquake magnitudes with <0.0004 annual rate of occurrence per 1,000 km^2. The specific values used in the simplified procedure for each bridge site are listed in appendix A.

Many bridges in the state were determined to have low liquefaction hazard based on non-site-specific information. For those sites with potential hazard, site-specific investigations were conducted using standard penetration equipment and the simplified procedure. Likewise, unless otherwise specified, the hammer energy ratio, ER_m, used in all of the analyses was assumed to be 50 percent.

Except for a few sites, the fines and clay contents (grain-size distributions) were estimated from soil classifications on borehole logs or from Atterberg limit data. Clean sand (AASHTO classification A-3) was assigned 5 percent fines, moderately silty sand (AASHTO A-1-a, A-1-b) was assigned 15 percent, and silty sand (A-2-4) was assigned 35 percent fines content. All silts and clays (A-4, A-5, A-6, A-7-5, and A-7-6) were assigned >36 percent fines. The assigned fines contents listed above were arbitrarily assumed to correspond with fines content curves on the standard CRR-$(N_1)_{60}$ plot in figure 21. The assumed fines contents should be conservative and provide smaller estimates of CRR than would be calculated if actual fines contents were available.

None of the borehole logs listed hydrometer results, so the clay contents were estimated using correlations with either the soil classification or Atterberg limits (where available). All soils listing clay as the major constituent (e.g. plastic clay, silty clay or sandy clay) and high plasticity silts were classified as non-liquefiable (they were assumed to have clay contents greater than 15 percent as stated in section 4.2). All other soils were assigned clay contents less than 15 percent.

All of the bridge sites that were screened using site-specific analyses are listed in appendix A along with the results from the analyses. To reduce the amount of calculation required, only one or two holes were selected from each site for liquefaction analyses. The boreholes with the greatest apparent potential for liquefaction were usually selected. One or two holes with liquefiable layers were sufficient to confirm a potential for liquefaction and the need for further investigation.

6.2 I-84 in Box Elder County

I-84 extends from the Idaho border to Tremonton where it merges with I-15. All but one of the bridge sites along this stretch of interstate were classed as having low liquefaction hazard and low priority for further study based on the groundwater map by Hecker and others (1988).

All the sites except I-84 bridge over Main St. Tremonton, are characterized by groundwater depths deeper than 9 m. I-84 over Main St. is characterized by nonliquefiable clay sediments to depths of 50 m. Thus, sites from the Idaho border to the junction with I-15 are Priority IV (low hazard and low priority for further investigation).

6.3 I-15 in Box Elder County

I-15 generally passes through the major population centers and parallels the major mountain fronts and the major faults as it traverses the state from the Idaho to the Arizona border. Near the Idaho border, the bridge sites lie in areas classed as low liquefaction potential by Anderson and others (1994a). I-15 passes through a narrow zone of high liquefaction potential where it crosses the Malad River north of Tremonton. From Tremonton south to the Weber County line, the highway crosses another moderately high to high zone. All of the bridges in the latter zones were analyzed using the site-specific penetration analysis outlined in section 4.3, and the results tabulated in appendix A.

Foundation investigations for seven bridge sites in Box Elder County were conducted using nonstandard penetration equipment. Five of these, Sta. 2373+28, Honeyville interchange, Calls Fort Rd, near Sta. 1946, and South Brigham City interchange (listed in appendix A with the prefix letter "U"), were investigated using a "Brandley Type U Soil Sampler". This sampler was driven by a 89 kg hammer dropped from a height of 61 mm. Due to the non-standard size of the sampler, hammer, and drop height, a conversion chart was used to estimate SPT blow counts. A copy of this conversion chart is reproduced in figure 30. The other two sites, I-15 at S.R. 83 (Brigham City to Corrine) and I-15 over Bear River, were tested by UDOT using a 64 mm OD split-spoon sampler driven with a 66 kg hammer dropped from a height of 76 cm. Figure 30 was also used to estimate SPT blow counts from these non-standard tests. All of the remaining sites were tested after 1960 using standard equipment.

Appendix A shows that all seven of the analyzed bridge sites are underlain by one or more layer of potentially liquefiable soil. These layers do not appear to be interconnected between sites. Figures 31 to 33 show depths and thicknesses of liquefiable layers beneath Priority I and II bridge sites in Box Elder County. The three sites at river crossings (I-15 over Malad River, I-15/I-84 over Malad River, and I-15 over Bear River) were assigned a higher hazard and priority because of the high potential for lateral spread. The shaded areas below 15 m indicate layers where liquefaction could possibly occur and where bearing capacity of deep foundations could be affected. These layers are too deep, however, to induce lateral spreads or other mass instability. Of the estimated 35 bridge sites on I-15 from Idaho to Weber County, 3 are Priority I, 24 are Priority II, and the remaining (about 8) are Priority IV. Appendix A lists the bridge sites with high priority for further study.

6.4 I-15 in Weber County

The entire stretch of I-15 in Weber County lies in zones of either moderate or high liquefaction potential (Anderson et. al., 1994a). All 19 of the I-15 Bridge sites in Weber County were analyzed using site-specific data. The borehole investigations for bridge sites were conducted using standard equipment and procedures. Results of these analyses are listed in appendix A. Possibly liquefiable layers, as shown in figures 34 and 35, underlie all of these sites. The I-15 Bridge over the Weber River is Priority I and the rest are classed Priority II.

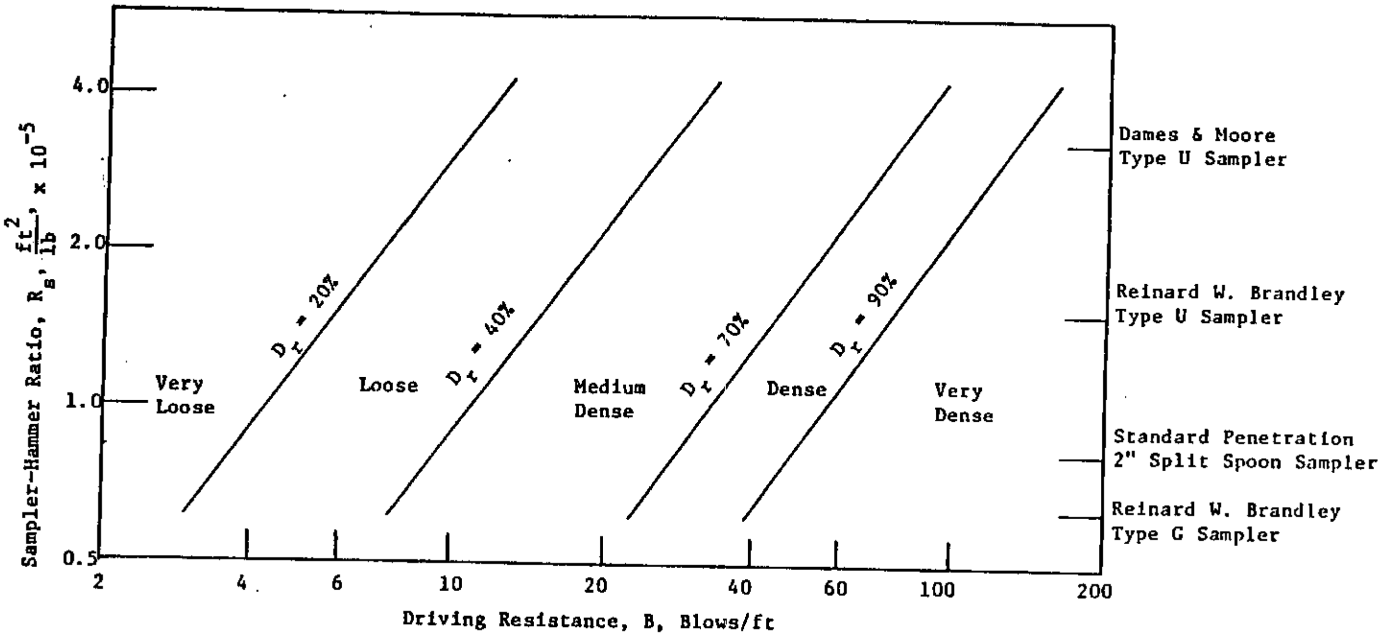
6.5 I-84 in Weber, Morgan, and Summit Counties

I-84 separates from I-15 near Ogden and runs southeast to Echo where it joins I-80. The routing of I-84 from I-15 to Uintah Junction crosses terrain mapped as moderate to high liquefaction potential by Anderson and others (1994a). Consequently, all seven bridges in Weber County were evaluated using penetration data. All but the Uintah Junction interchange and the nearby crossing over the Weber River are underlain by potentially liquefiable soil layers, as shown in figure 36.

The remainder of the I-84 freeway was screened using the shallow groundwater maps compiled by Hecker and others (1988). The shallow groundwater map indicates that the only areas with a water table less than 9 m are located along the Weber River flood plain from Mountain Green to Morgan, and from Devil's slide to Henefer. No borehole data could be found for the bridge sites at Peterson, Stoddard, and Morgan interchanges. These sites were classified as potentially hazardous and assigned a high priority (Priority II) for further study due to the shallow groundwater conditions near rivers (Hecker et. al., 1988). Figure 36 delineates the liquefiable layers beneath the I-84 Bridge sites. The associated analyses confirmed the presence of liquefiable layers and a high priority for further investigation. Of the estimated 22 bridge sites on I-84, 1 was assigned Priority I, 11 were assigned Priority II, and 10 were assigned Priority IV. Appendix A contains a listing of these sites with their assigned priorities.

6.6 I-15 in Davis County

I-15 in Davis County crosses zones classed as moderate and high liquefaction potential over most of its route except for two narrow zones of low potential near the North Layton interchange and Wood Cross exit (approx. 2600 South in Bountiful) (Anderson et. al., 1994b). The only interstate bridge in Davis County that was classed as Priority IV, based on the liquefaction hazard maps, was the Woods Cross exit. The North Layton interchange bridge, though located in the zone of low hazard, was analyzed using the simplified procedure because of the shallow water table. The site was found to have potentially liquefiable layers as indicated in figure 37. No borehole data could be found for the Hwy. 89 crossing over I-15. This site was classed as Priority III, insufficient information for analysis but moderate priority for future investigation.



$$R_s = \frac{D_o^3 - D_i^3}{144\ WH}$$

D_o = Outside Diameter of Sampler, inches

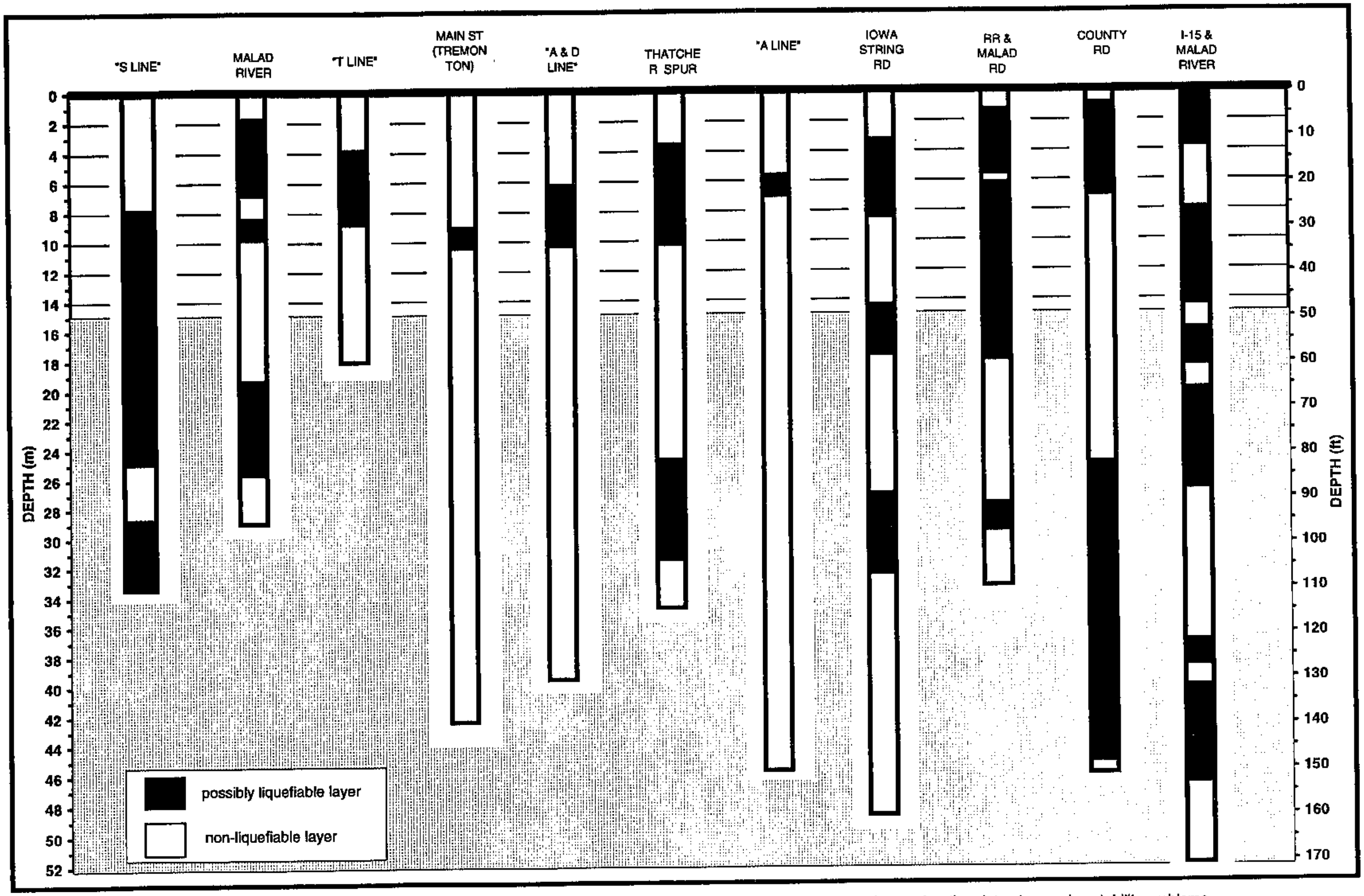
D_i = Inside Diameter of Sampler, inches

W = Weight of Hammer, pounds

H = Height of Drop, inches

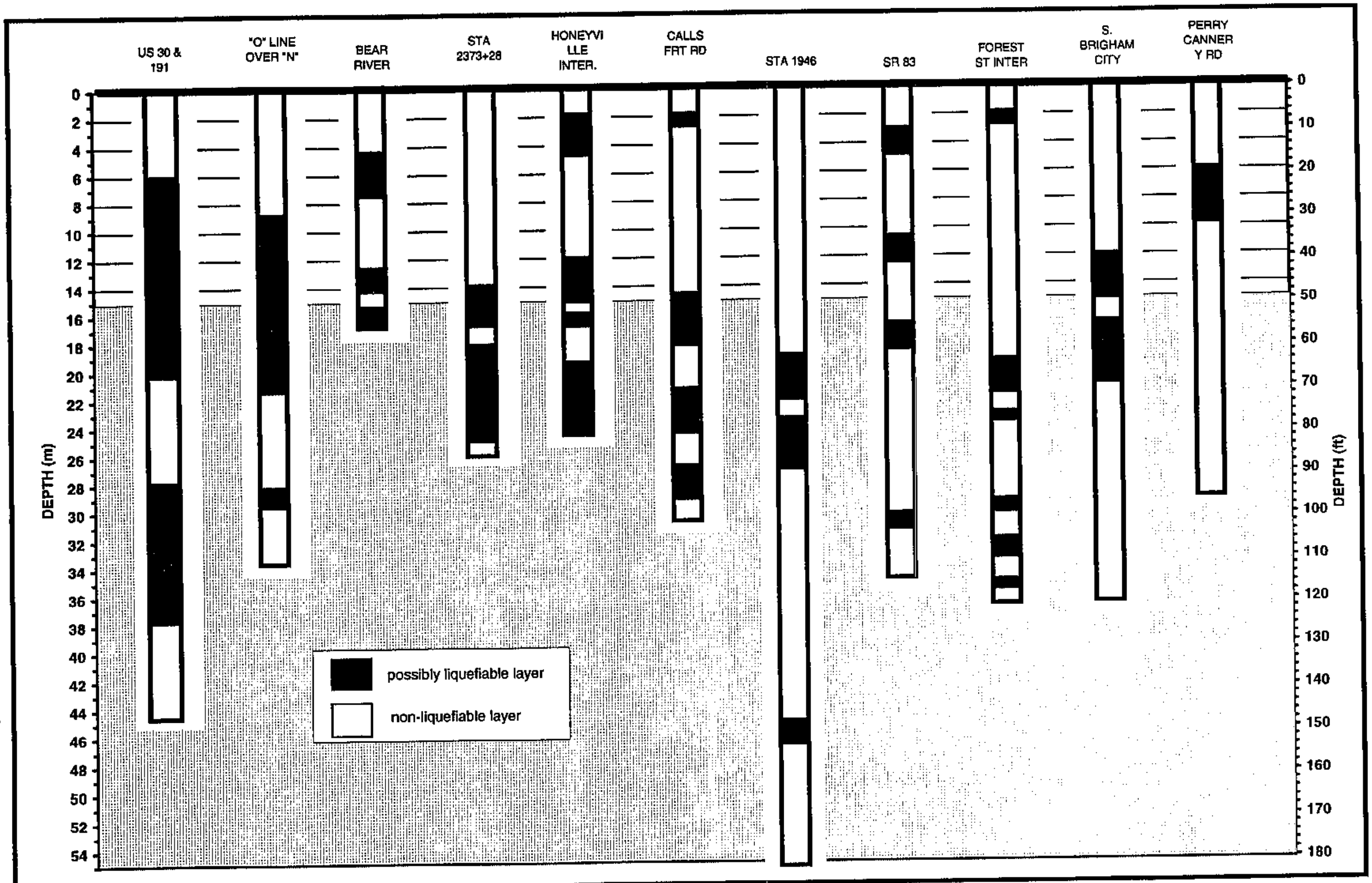
D_r = Relative Density, %

Figure 30: Penetration resistance conversion relationship for cohesionless sand and silts (after Anderson et. al., 1986)



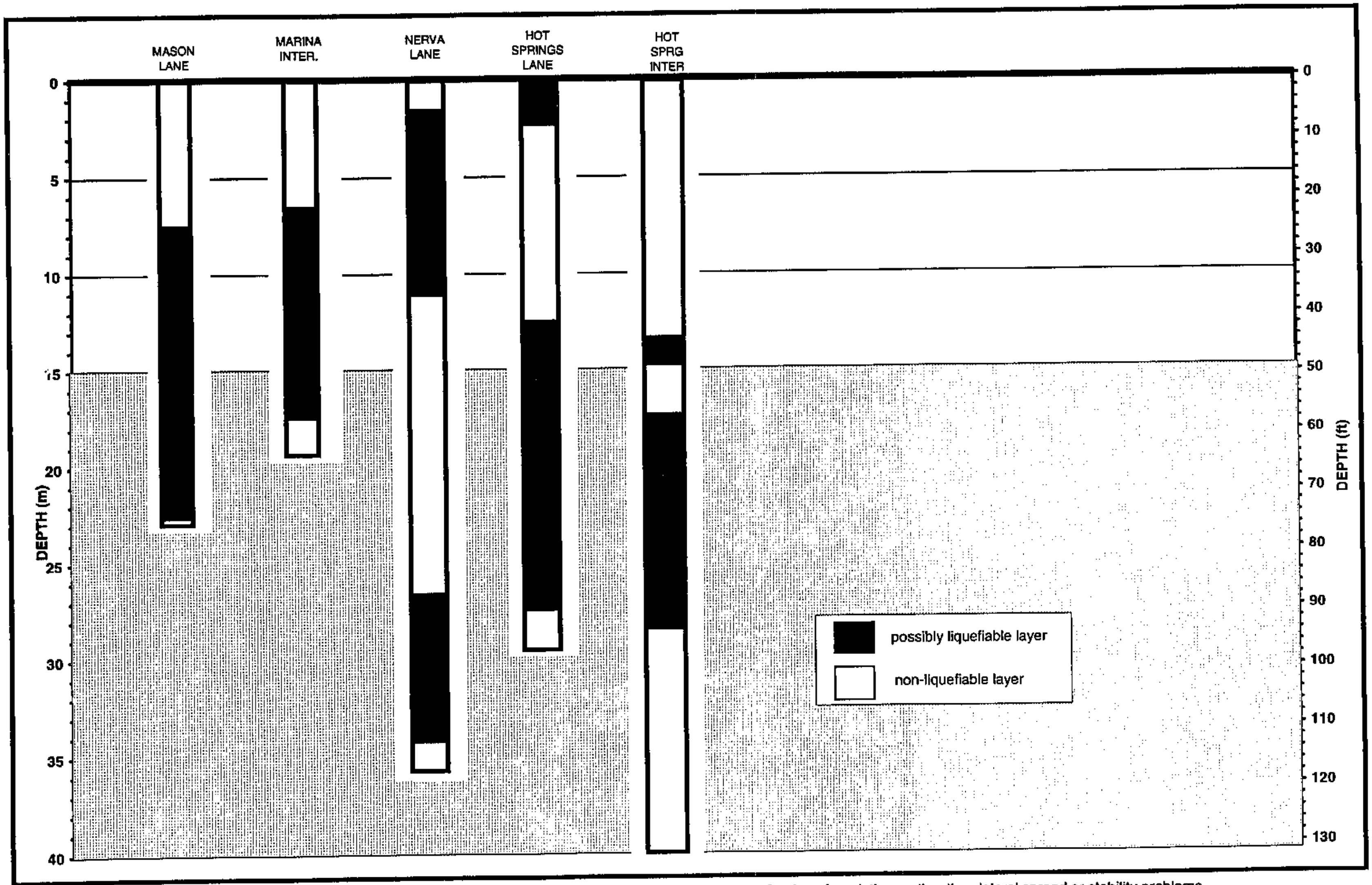
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 31: Logs for I-15 in Box Elder County from Idaho border to Tremonton showing potentially liquefiable layers in soil profile.



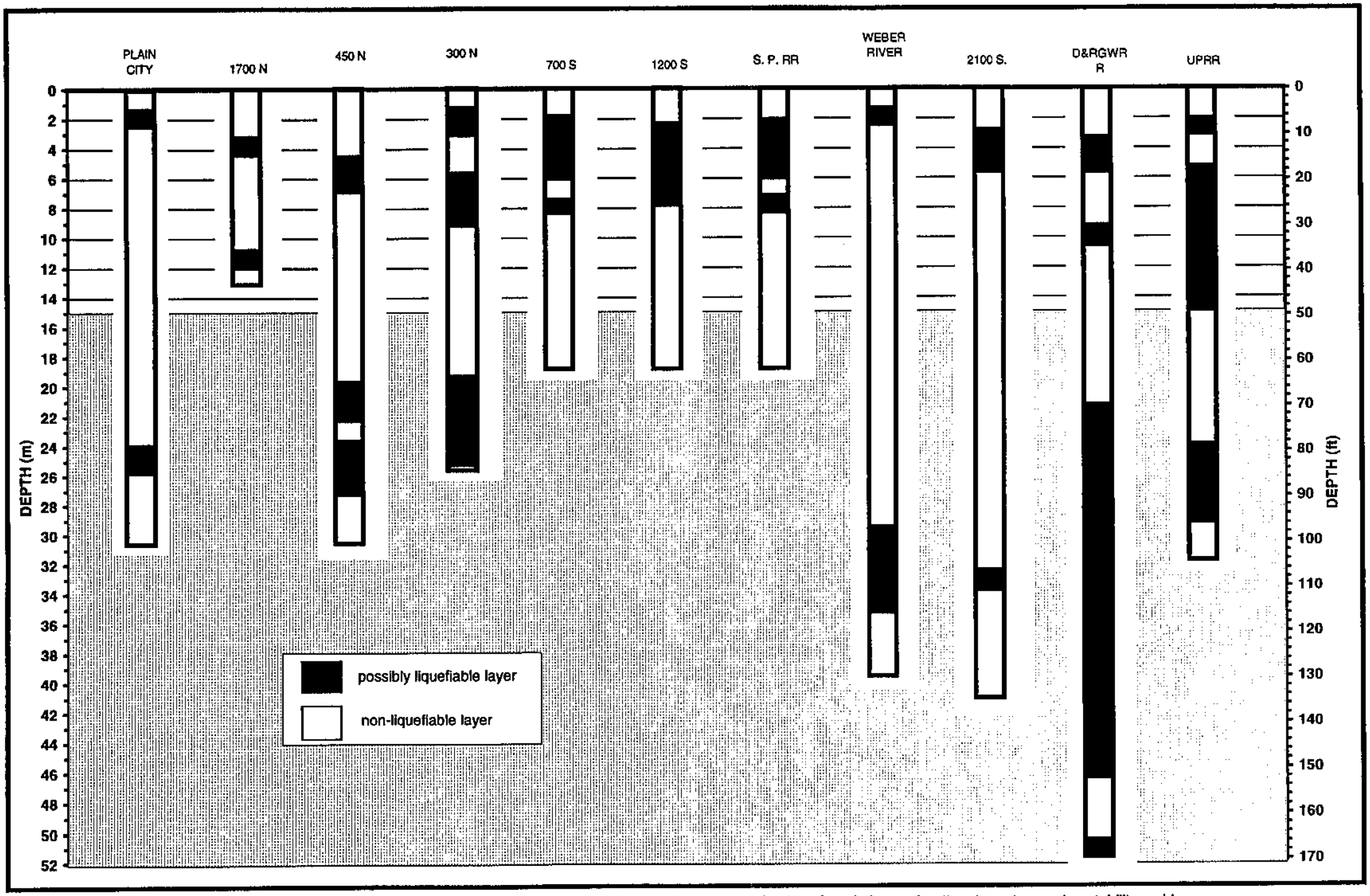
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 32: Logs for I-15 in Box Elder County from Tremonton to Brigham City showing potentially liquefiable layers in soil profile.



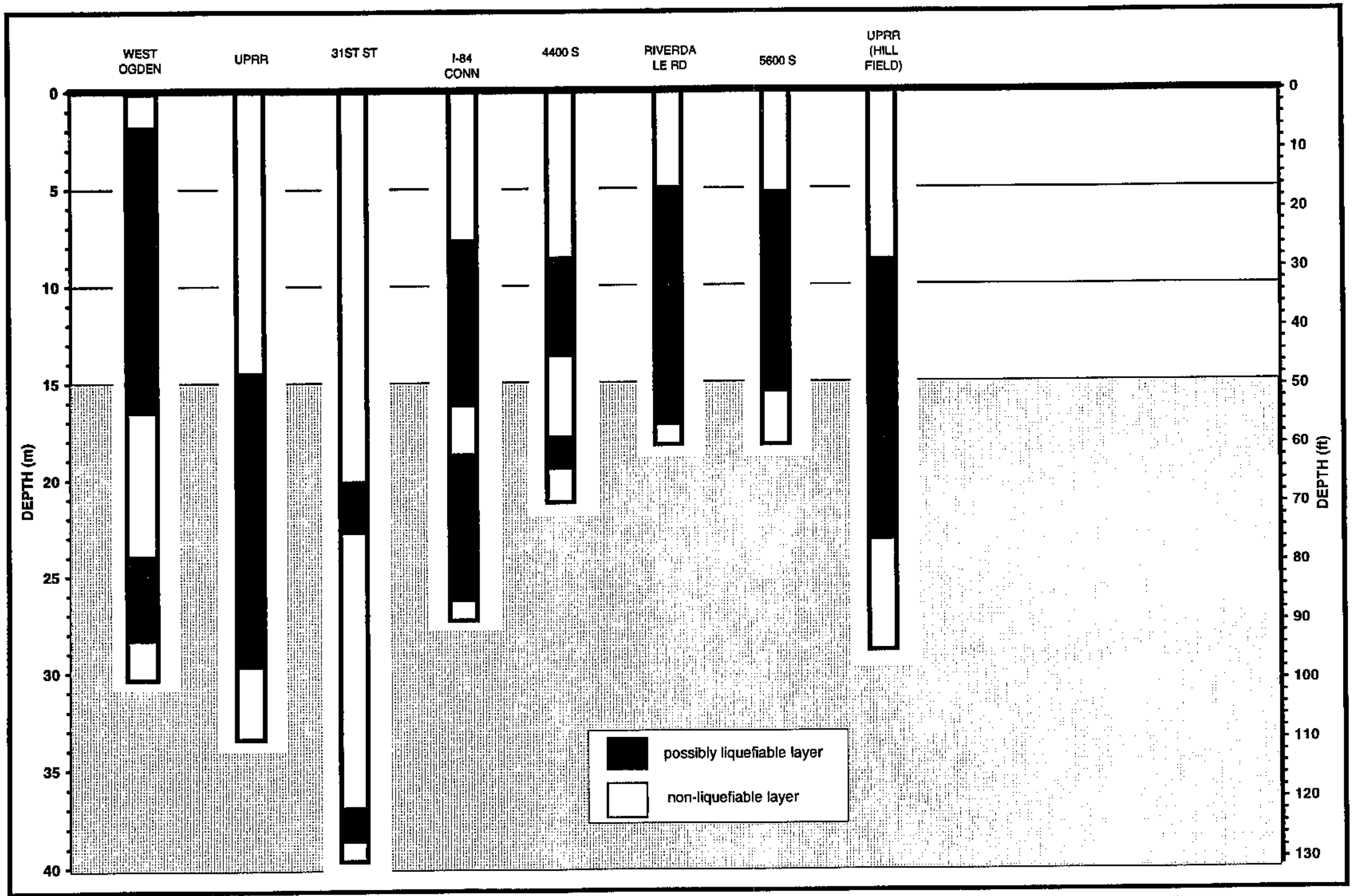
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems. Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 33: Logs for I-15 in Box Elder County from Brigham City to Weber County showing potentially liquefiable layers in soil profile



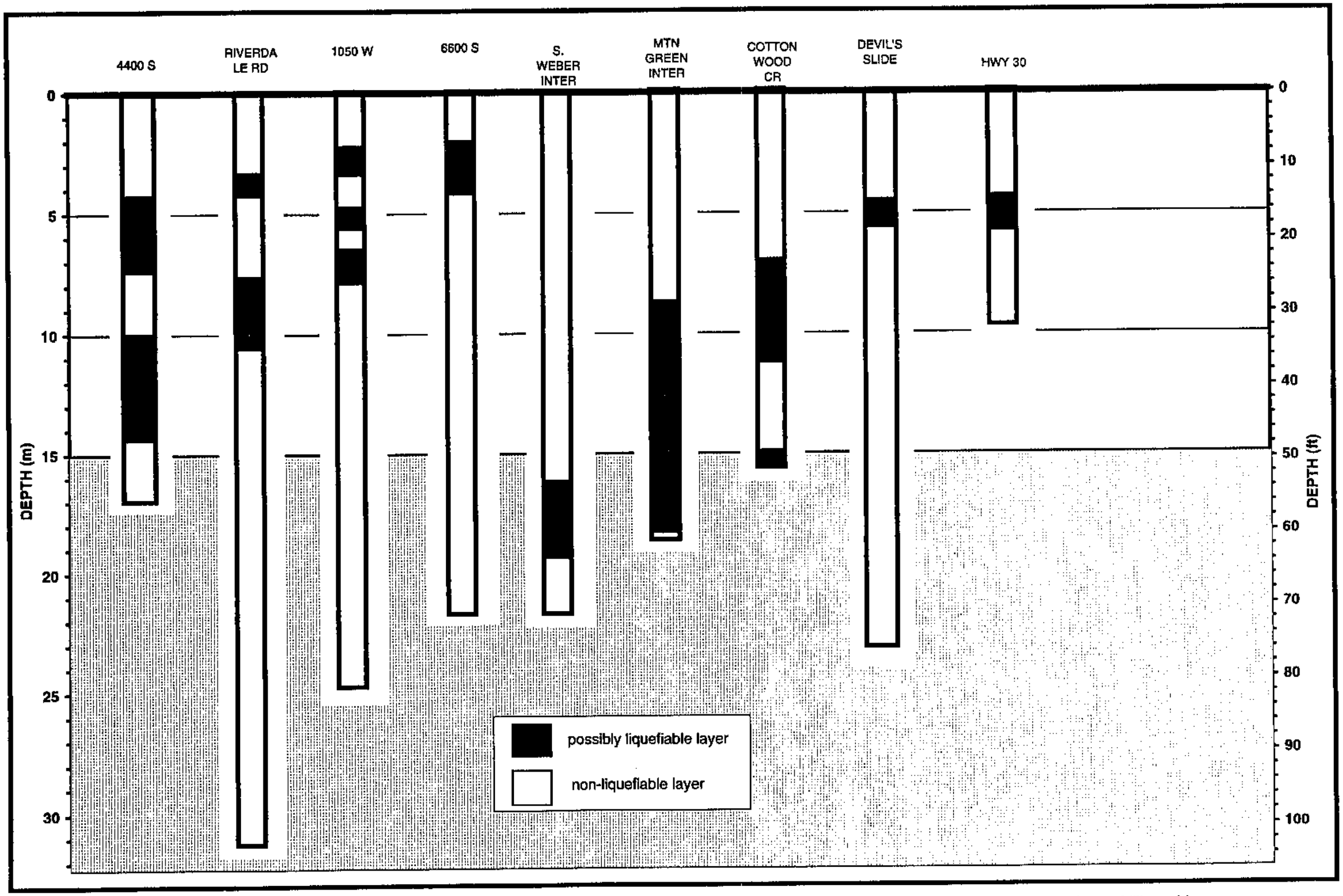
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 34: Logs for I-15 in Weber County from Box Elder County to Ogden showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 35: Logs for I-15 in Weber County from Ogden to Davis County showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 36: Logs for I-84 from Weber County to I-80 junction showing potentially liquefiable layers in soil profile.

All Davis County bridge sites were investigated using standard equipment except for sites at Pages Lane, 1000 North, 400 North, 500 South, 1500 South in Bountiful, and the bridge at 1100 West in North Salt Lake. These six sites, listed with the prefix "U" in appendix A, were tested using a Brandley sampler as explained in section 6.3. SPT blow counts were estimated using figure 30.

The results of the site-specific analyses are shown in figures 37 and 38. These figures indicate that all of the sites evaluated are underlain by potentially liquefiable layers. These layers range in thickness from less than 1 m to tens of meters. Of the 25 bridge sites on I-15 in Davis County, none are Priority I, 23 are Priority II, 1 is Priority III, and 1 is Priority IV. Data and results from the analyses of these sites are listed in appendix A.

6.7 I-80 from I-15 to the Wyoming Border

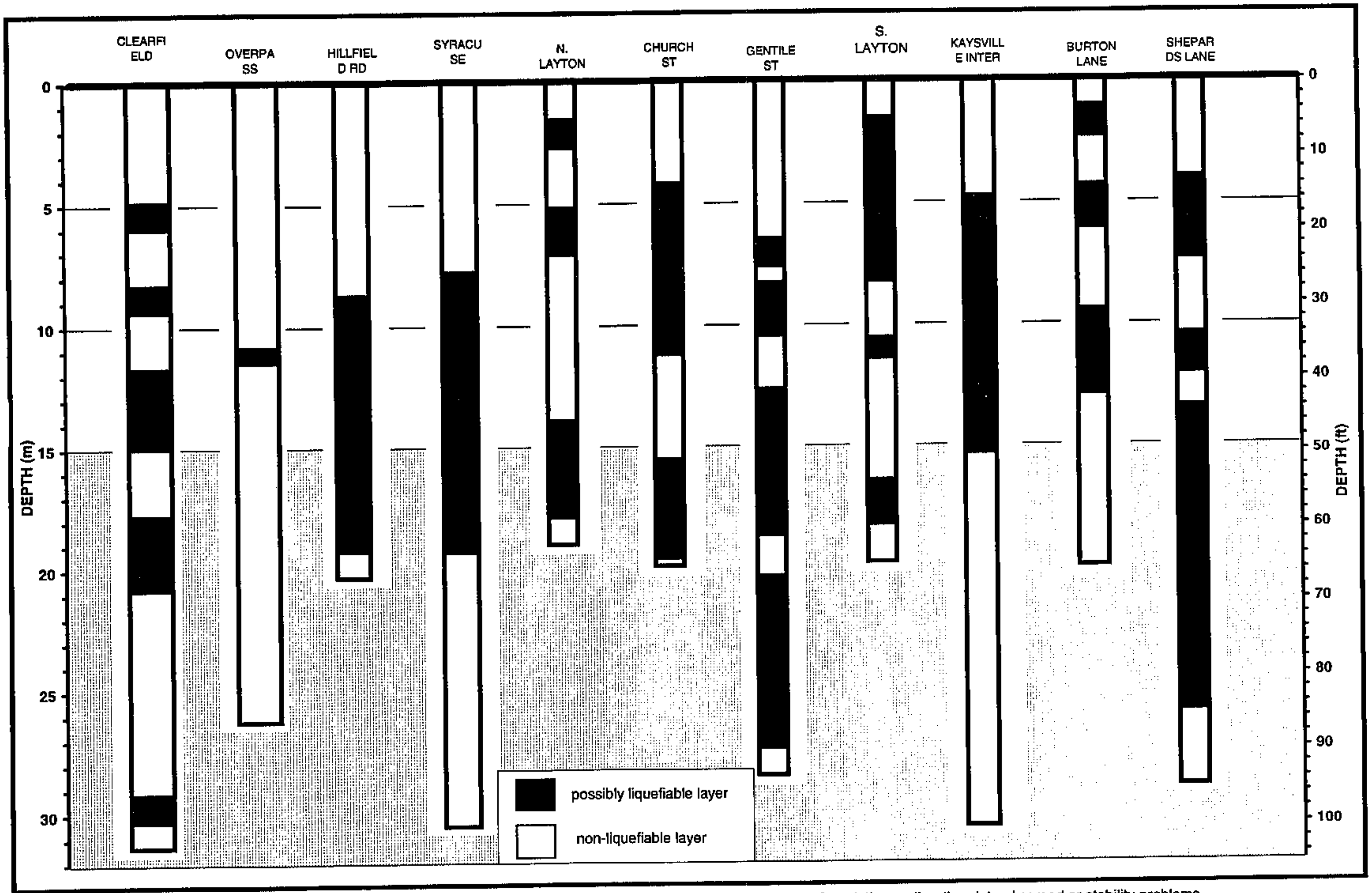
This section of interstate runs from the junction with I-15 in Salt Lake County eastward to the Wyoming border. Based on liquefaction potential maps (Anderson et. al., 1994d; Anderson et. al., 1994e), all of the bridge sites in Salt Lake County are located in zones of moderate to high liquefaction hazard. These sites include overcrossings at 200 West, UPRR, West Temple, Main St., State St., 300 East, 500 East, 700 East, 900 East, and 2300 East. A site-specific evaluation, however, indicated that the water table at 2300 East was too deep (greater than 15 m) for liquefaction to be a hazard. Site-specific analyses indicate that the remaining sites are underlain by liquefiable layers as noted on figure 39.

Based on the maps by Anderson and others (1994e), the Park City interchanges (Kimball Junction, "D" Line, and Silver Creek Junction) lie in areas of low to very low liquefaction potential. However, the ground water table is high around the three bridge sites (Hecker et. al., 1988). Analysis of site-specific data from these sites confirmed the low liquefaction potential. The remaining sites east of Park City have deep (greater than 9 m) groundwater so they were classed as low priority for further study (Priority IV).

Of the estimated 30 bridge sites along I-80 from Salt Lake County to Wyoming, 9 were Priority II (confirmed existence of liquefiable layers and high priority for further study) and 21 were Priority IV (low liquefaction hazard and low priority for future study).

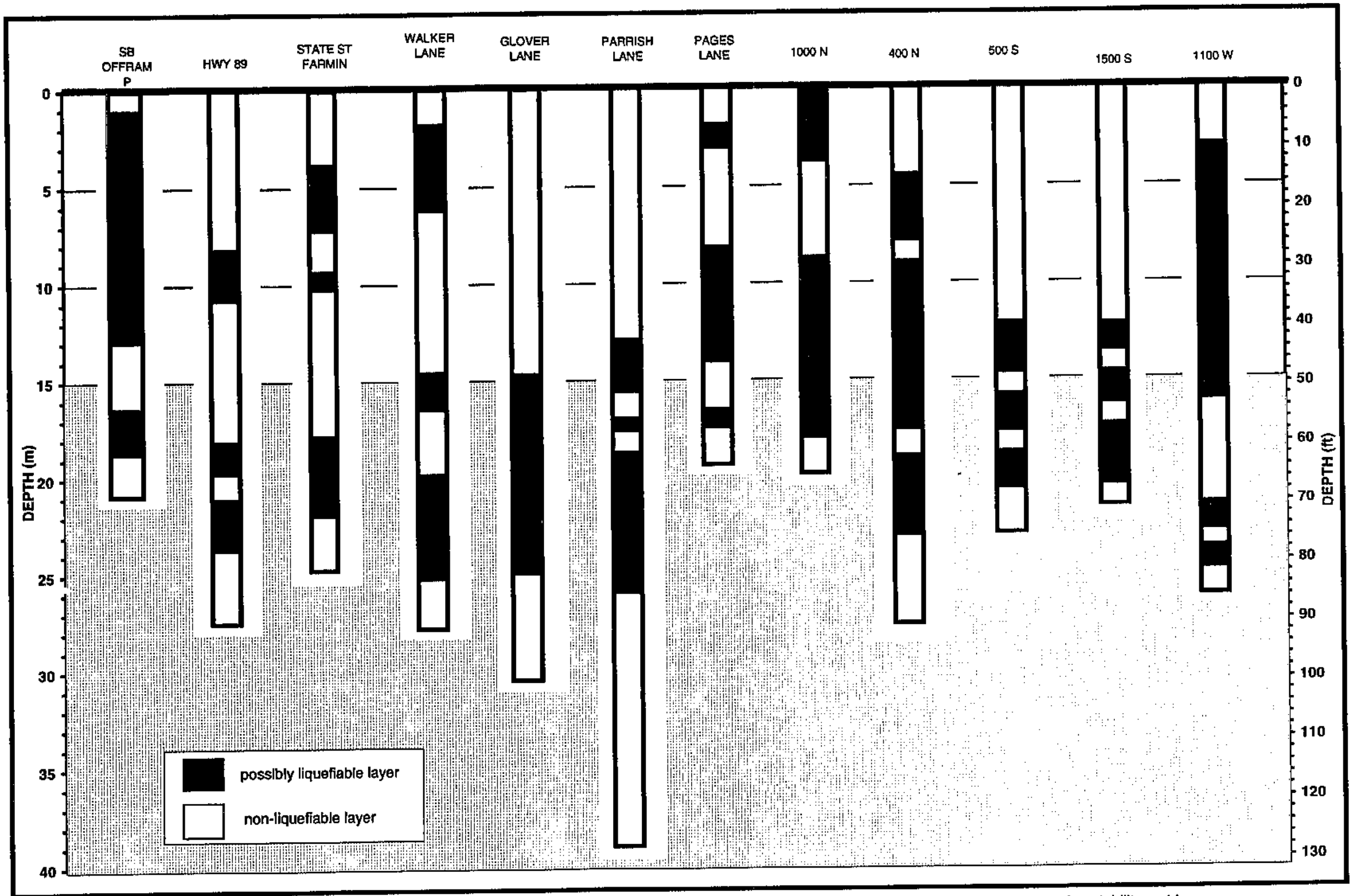
6.8 I-80 from I-15 to the Nevada border

Interstate 80 extends westward from the junction with I-15 in Salt Lake County to Nevada. Nearly all of the roadway alignment lies on Lake Bonneville deposits (Currey and others, 1984) which are considered to be potentially liquefiable based on the geologic age of the sediments. For Salt Lake County, Anderson and others (1994d) show that I-80 traverses areas of high liquefaction potential. The Saltair interchange and the bridge sites at 200 South, Navajo St., UPRR, and 1000 West had inadequate borehole information to perform site-specific analyses. Because those four sites are located on sediments which Anderson and others (1994d) classify as



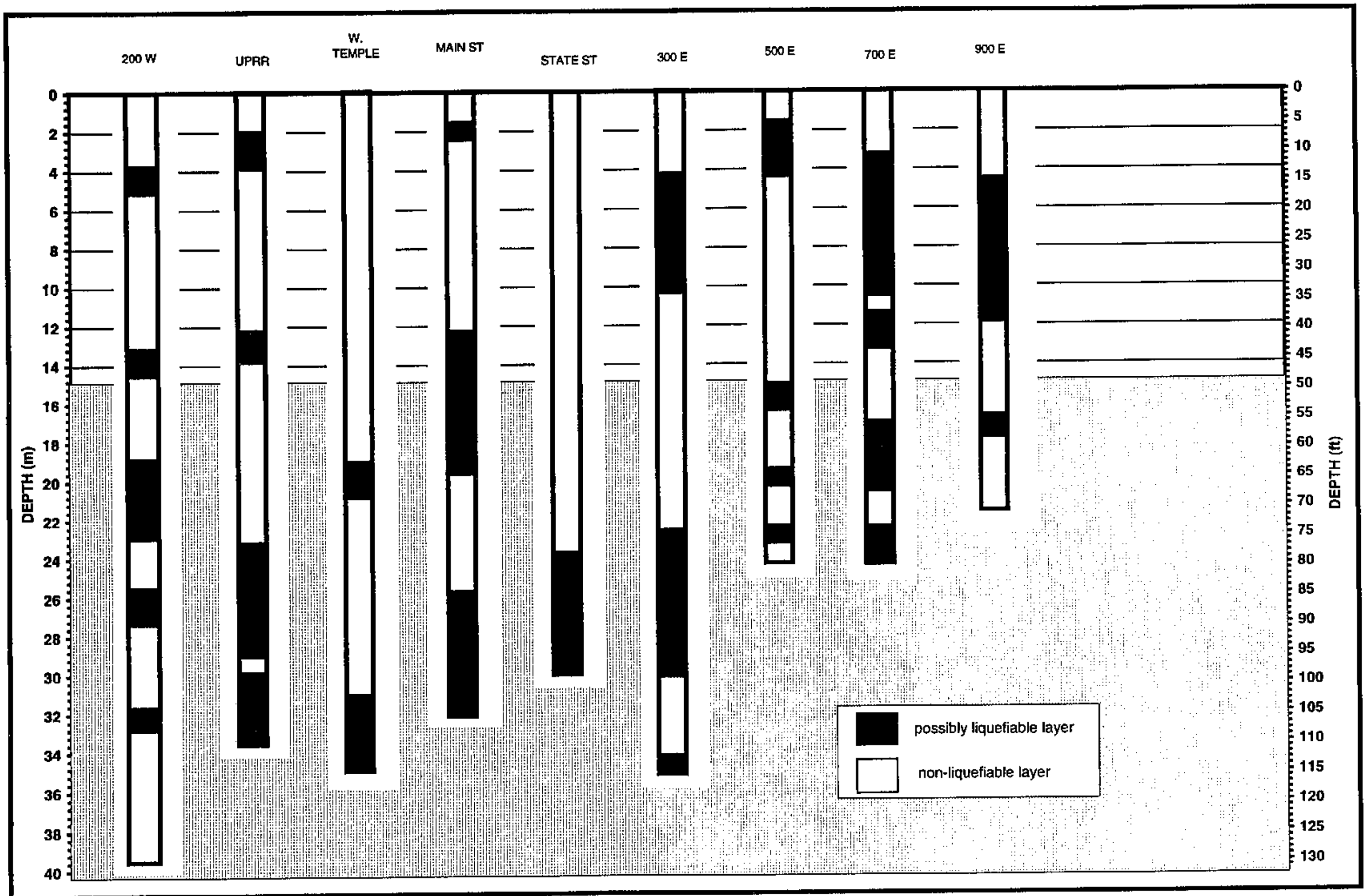
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 37: Logs for I-15 in Davis County from Weber County to Kaysville showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 38: Logs for I-15 in Davis County from Farmington to Salt Lake County showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems. Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 39: Logs for I-80 in Salt Lake County from I-15 to 900 East showing potentially liquefiable layers in soil profile.

having a high liquefaction potential, they are classed as possibly hazardous and given Priority III for further study. Of the remaining 11 sites in Salt Lake County, one is Priority I (I-80 over Jordan River) and 10 are Priority II. These have very high and moderately high priority for further study. Figure 40 delineates the liquefiable layers beneath these 11 sites.

In Tooele County, when the penetration data were evaluated, all of the sites except the Clive and Aragonite (Low) interchanges, I-80 under county road-station 68+60, and I-80 under access road- station 130+51 were found to be underlain by liquefiable sediments. Clive and Aragonite interchanges have deep groundwater tables and the county and access roads have clay sediments to depth. Figure 41 delineates the liquefiable sediments at the bridge sites in Tooele County. Of the 15 bridge sites in Tooele County, 11 sites are Priority II and 4 are Priority IV.

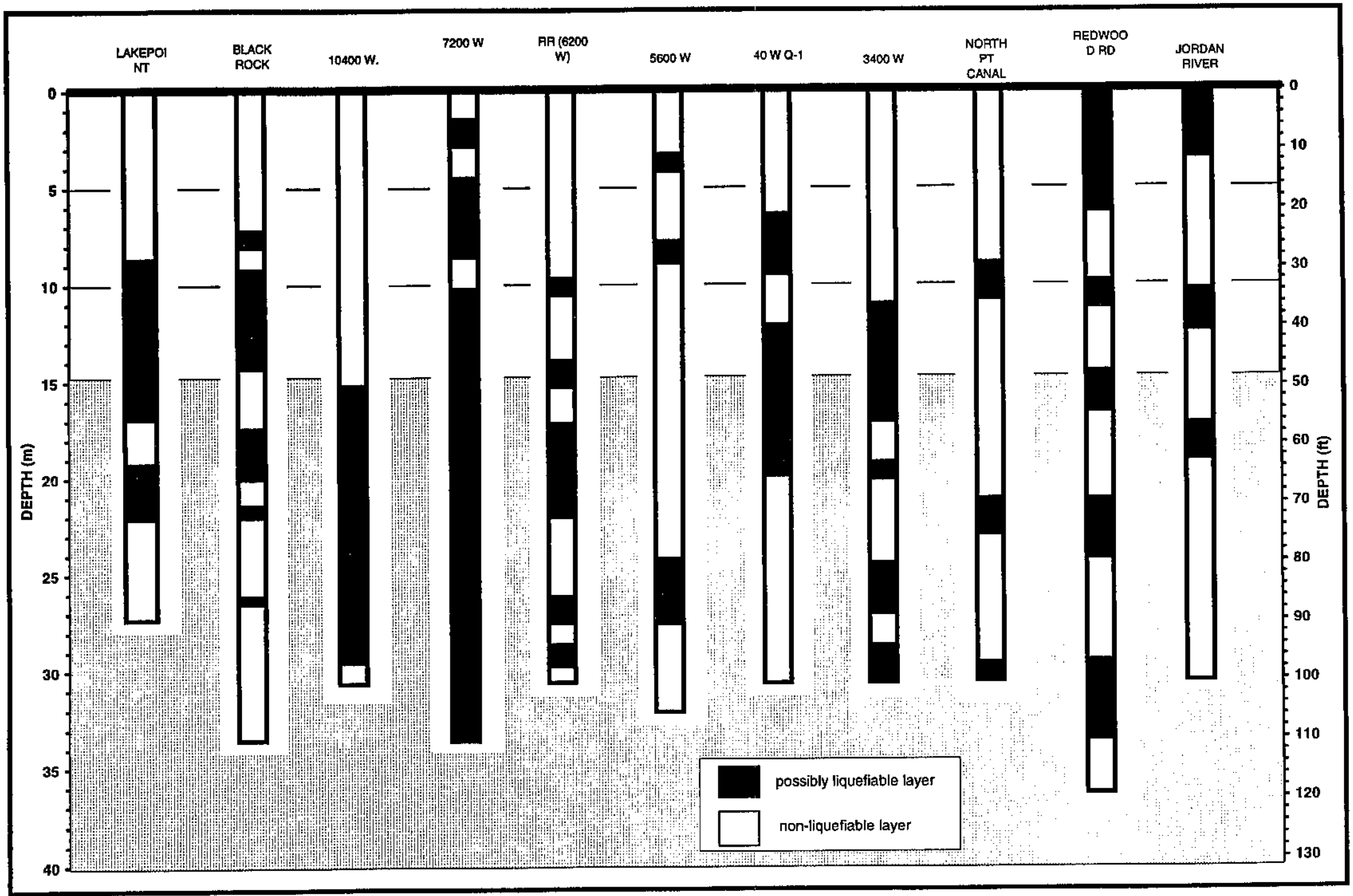
6.9 I-15 in Salt Lake County Using Historical SPT Data

The entire I-15 corridor in Salt Lake County is located in areas zoned by Anderson and others (1994d) as either high or moderate liquefaction potential except at the south end of the county (near Point of the Mountain) where the liquefaction potential is low. The majority of the borehole logs for I-15 in Salt Lake County were drilled prior to 1965. These investigations were performed to provide information for design and construction of the original interstate highway structures. The I-15 corridor is divided into three sections based on the interstate segment and past subsurface investigation contracts. Proceeding from north to south, the segments are from Davis County to the I-80 interchange (about 2600 South), from 2700 South to 5900 South, and from 6400 South down to 14600 South (Point of the Mountain).

6.9.1 Davis County to I-80 Interchange (Porter, Urquhart, McCreary and O'Brian Borings)

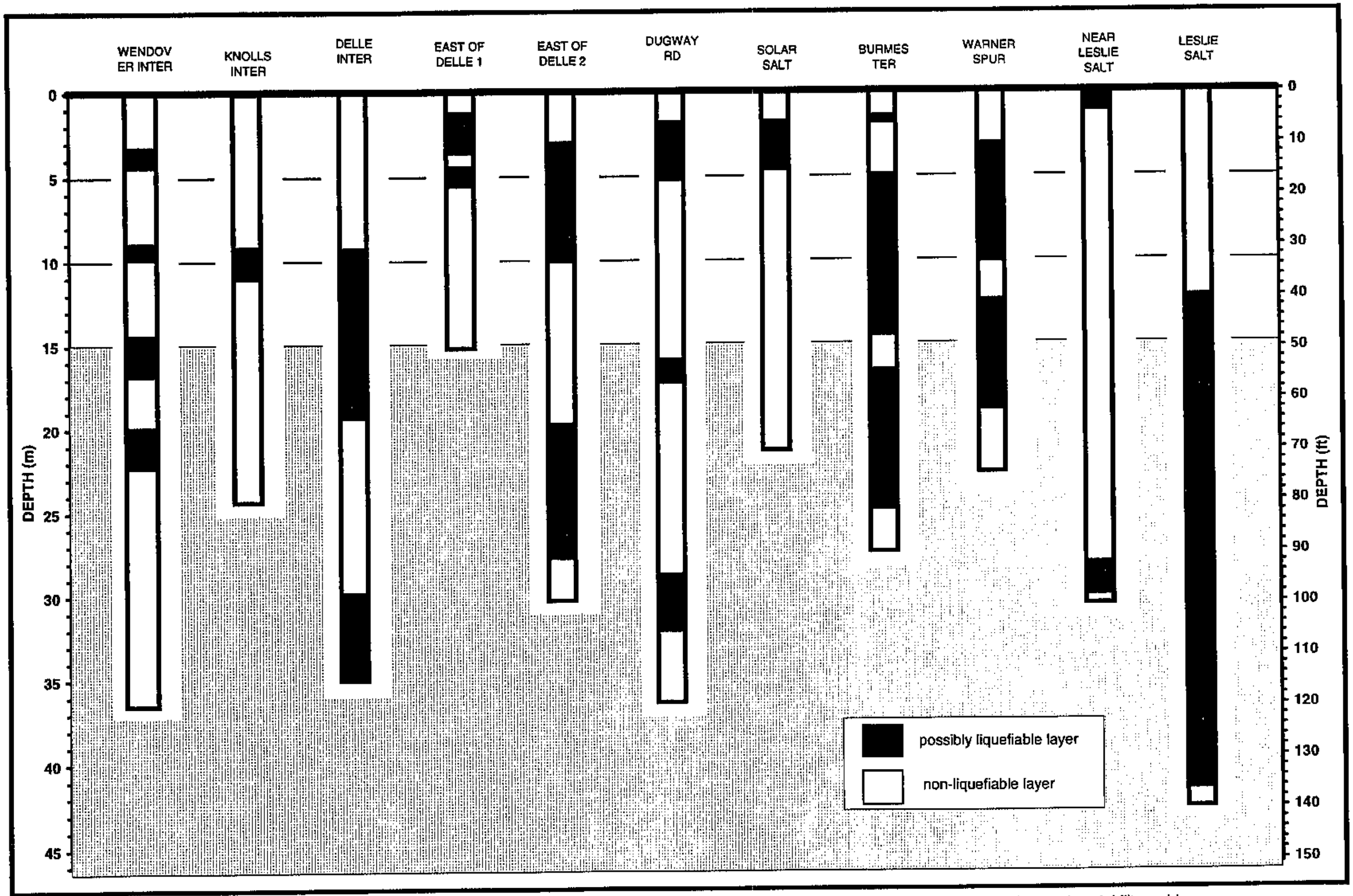
The liquefaction potential maps (Anderson et. al., 1994d) show that two sites, Beck St. overpass, and 800 West exit, are located in zones of high liquefaction potential. These sites are therefore classed as Priority III. No data was available for the two bridge sites from the Davis County line to North Temple so they were not analyzed using site-specific data.

The boreholes designated in appendix A by the prefix "DH" are characterized by higher-quality standard penetration data. SPT blow counts had to be estimated for the boreholes listed with the prefix "P" in appendix A (drilled by the consulting firm of Porter, Urquhart, McCreary and O'Brian in the late 1950's), because they were investigated using non-standard equipment. As noted on figures 42 and 43, one or more layers of apparent liquefiable sediment lie beneath each bridge site between North Temple and the I-80 interchange. Because of the pervasiveness of susceptible layers, some of these layers are likely to be interconnected between bridge sites and may be rather continuous beneath the area. All 15 of the sites south of North Temple have confirmed existence of liquefiable layers and thus high priority for further investigation (Priority II).



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 40: Logs for I-80 in Salt Lake County from Lakepoint to I-15 junction showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 41: Logs for I-80 in Tooele County from Nevada border to Salt Plants showing potentially liquefiable layers in soil profile.

6.9.2 2700 South to 5900 South (Reinard W. Brandley Borings)

The second section of the I-15 corridor extends from 2700 South down to 5900 South. With the exception of two holes, all of the borings in this section were drilled and tested with non-standard equipment. The two exceptions were hole DH-12 at 5900 South and DH-11 at 3300 South, both of which were drilled in the 1990's. The remaining borings, those shown on appendix A with the prefix letter "U", had samples and blow counts taken using the "Brandley Type U Soil Sampler". To utilize this data, SPT blow counts were estimated using figure 30.

The liquefaction resistance analyses indicated that susceptible sediments lie beneath each of the bridge sites from 2700 South to 5900 South as noted on figures 43 and 44. Therefore, the nine bridges are prioritized as Priority II. The susceptible layers in this segment appear to be intermittent with no consistent pattern indicative of a single pervasive liquefiable layer.

6.9.3 I-215 Interchange to 14600 South (Fuhriman, Hodson, and Rollins Borings)

The segment from the I-215 interchange to 14600 South (Point of the Mountain) contains some of the more complete and reliable blow count data. Standard hammers and samplers were used for penetration tests in this segment of the corridor so none of the blow counts needed to be corrected. The site at 14600 South is located in a zone of low liquefaction potential (Anderson et. al., 1994d) so it was classed as Priority IV.

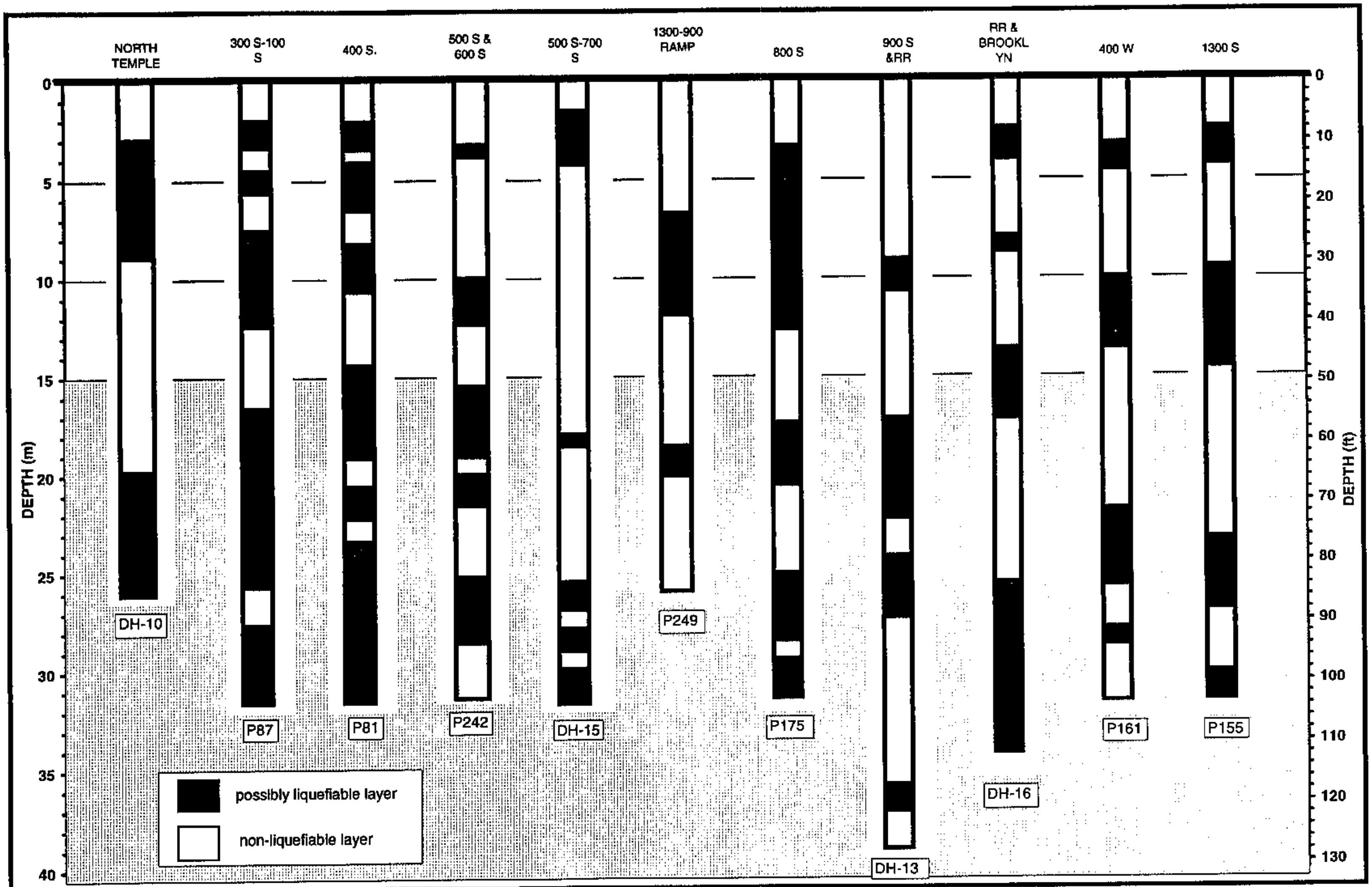
Results from the liquefaction resistance analysis for the segment from the I-215 interchange to 14600 South are shown in figure 44. All of the boreholes examined in this segment, with the exception of 14600 South which is Priority IV, contain some potentially liquefiable layers and are thus classed as Priority II. Of the 36 bridge sites in the I-15 corridor through Salt Lake County, 33 are classed as Priority II, 2 as Priority III, and 1 as Priority IV.

6.10 Results of Integrated Liquefaction Evaluation Procedure by Gilstrap

Samuel D. Gilstrap, an undergraduate student assigned to assist with this analysis, conducted the analyses and prepared much of the text reported in this section. This assistance is gratefully acknowledged. Both standard (SPT) and cone (CPT) penetration data were used in an integrated analysis, hereafter referred to as the "integrated analysis". For this study, the integrated analysis utilized state-of-the-art liquefaction analysis procedures and high quality subsurface data from preliminary subsurface investigations for the reconstruction of I-15 in Salt Lake County.

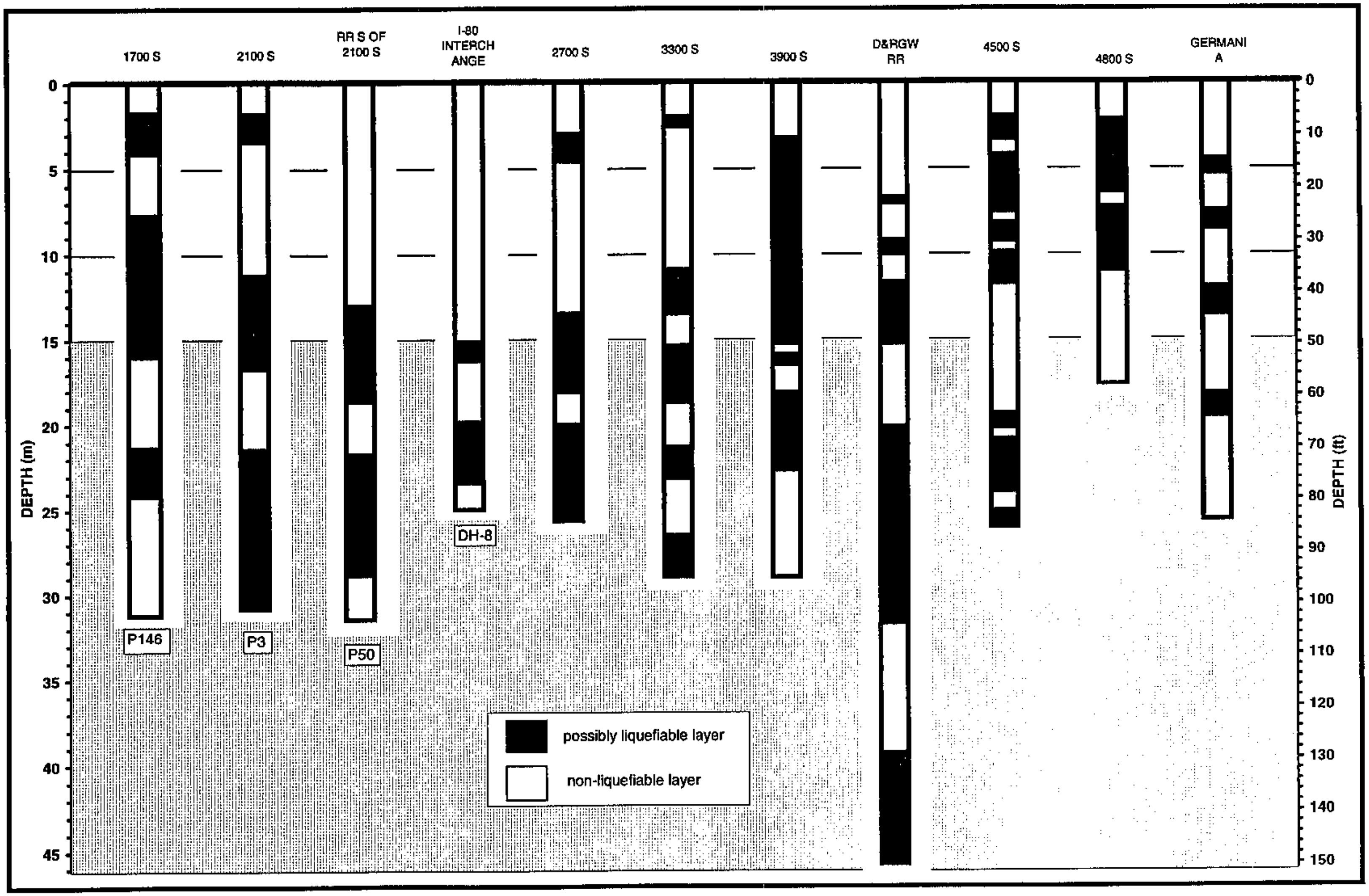
6.10.1 Summary of Liquefaction Logs

The results from the integrated study are plotted on figures 45 through 48 which show depths and thicknesses of possibly liquefiable layers. Non-liquefiable layers and potentially sensitive clays are also identified on the figures. The shading divides the logs into two important zones. The unshaded upper 15 meters denotes the depth in which liquefaction is most likely to influence embankment and foundation stability. The shaded zone below 15 meters indicates a



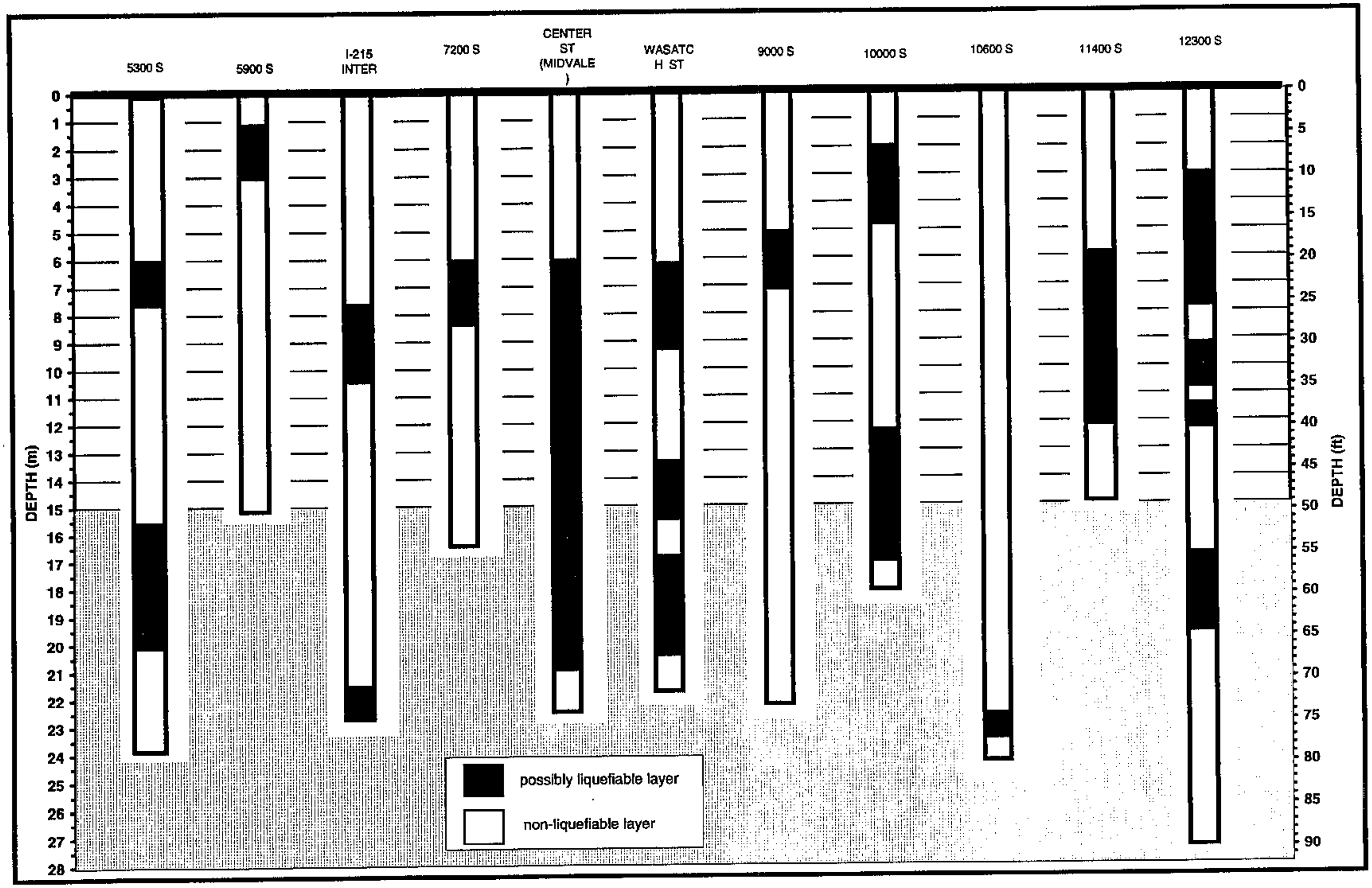
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 42: Logs for the I-15 corridor from Davis County to 1300 South showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems. Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 43: Logs for the I-15 corridor from 1700 South to Germania St. showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems. Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 44: Logs for I-15 corridor from 5300 South to 12300 South showing potentially liquefiable layers in soil profile.

zone which liquefaction would most likely only affect load capacity of deep foundations. Possibly sensitive clay layers are also noted on these plots. The liquefaction potential of sediments in the shaded area (deeper than 15 m) are more uncertain because the prediction procedure used is not well validated for depths greater than 15 meters.

6.10.2 Spreadsheets

Tabulated data and results for all bridge sites are contained on the discs in appendix C. The spreadsheets developed for this study directly apply the integrated liquefaction hazard evaluation procedure discussed in section 4.4. Each spreadsheet in appendix C lists the following information:

a) Earthquake magnitude and peak acceleration.

b) Input data from CPT and SPT logs and laboratory tests.

c) Calculated values for various factors developed in the analyses, including calculated factors of safety.

d) Notes from the CPT and SPT logs identifying the location, hole or sounding, ground surface elevations, water table depth and horizontal distance between companion borings and soundings.

6.10.3 Tables of Results for Individual Bridge Sites

The discs located in appendix C contain a summary of results for each bridge site plus a comparison with the results from the previous liquefaction analysis. These tables include the following information:

a) Factor of safety based on CPT analysis (discussed in section 4.4).

b) Factor of safety based on SPT analysis (discussed in section 4.4).

c) Soil log taken directly from SPT driller's log.

d) Liquefaction log from this study.

e) Liquefaction logs from previous study (sites listed in section 6.9).

6.10.4 Comparison of Integrated Against Previous Analysis

The integrated analysis had the advantage of more complete and more recent data than the previous study. The integrated analysis used CPT, SPT, and laboratory data from subsurface investigations, while the previous analysis relied on inadequate and incomplete SPT and laboratory data taken with non-standard equipment. Thus, the integrated analysis did not apply additional conservatism due to the lack of data or poor quality data to the extent of the previous analysis.

The integrated analysis created a continuous liquefaction log. The results of this integrated analysis yielded the following results:

a) The amount of material classed as liquefiable was reduced 54 percent for the entire corridor.

b) Liquefiable sediments underlay Eighty percent of the bridge sites.

c) Conversely, the amount of material classed as liquefiable increased at 20 percent of the bridge sites.

6.10.5 Discussion of Result by Freeway Segment

46th North to South Temple (figure 45)

This segment is underlain primarily by clayey soils with occasional sand layers. Several thin layers of liquefiable sediment were detected in the upper 15 meters; the cumulative thickness of liquefiable material ranges from 0.5 m to 2 m. At depths between 13 m and 20 m liquefiable layers up to 3 m thick were encountered. These layers are likely too deep to adversely affect embankment stability, but could affect pile load capacity. The liquefiable layers through this segment have a low to moderate certainty of liquefaction due to I_c values ranging between 2.05 and 2.65. At the 6th North site, a potentially sensitive clay layer 1 m thick was detected 2 m below the ground surface. This layer could adversely affect embankment stability.

2nd South to 13th - 9th South Off-ramp (figure 45)

The upper 15 m of segment 2 is primarily composed of clayey soils with 2 m to 3 m thick liquefiable sand layers between 2 m and 6 m deep at most bridge sites. This sandy layer has a moderate to high certainty of liquefiability with most I_c values below 2.40. A layer of potentially sensitive clay, approximately 0.5 m to 1.5 m thick, lies below the sand layer at some sites. The liquefiable layer and the sensitive clay layer may influence embankment stability or lead to lateral spread or ground settlement. Between depths of 14 m and 20 m, the site is underlain by a nearly continuous 2 m to 4 m thick liquefiable layer which may affect the load capacity of deep foundations. Other sites in this segment contain only sporadic thin liquefiable layers which are of much less concern to site and foundation stability.

8th South to RR south of 21st South (figure 46)

This segment has a 2 m to 6 m thick rather continuous liquefiable layer in the upper 15 m beneath most bridge sites which may affect embankment stability. The RR & Brooklyn Bridge site has the greatest potential with 4 m of liquefiable soil in the upper 6 m of the soil profile.

LIQUEFACTION LOGS

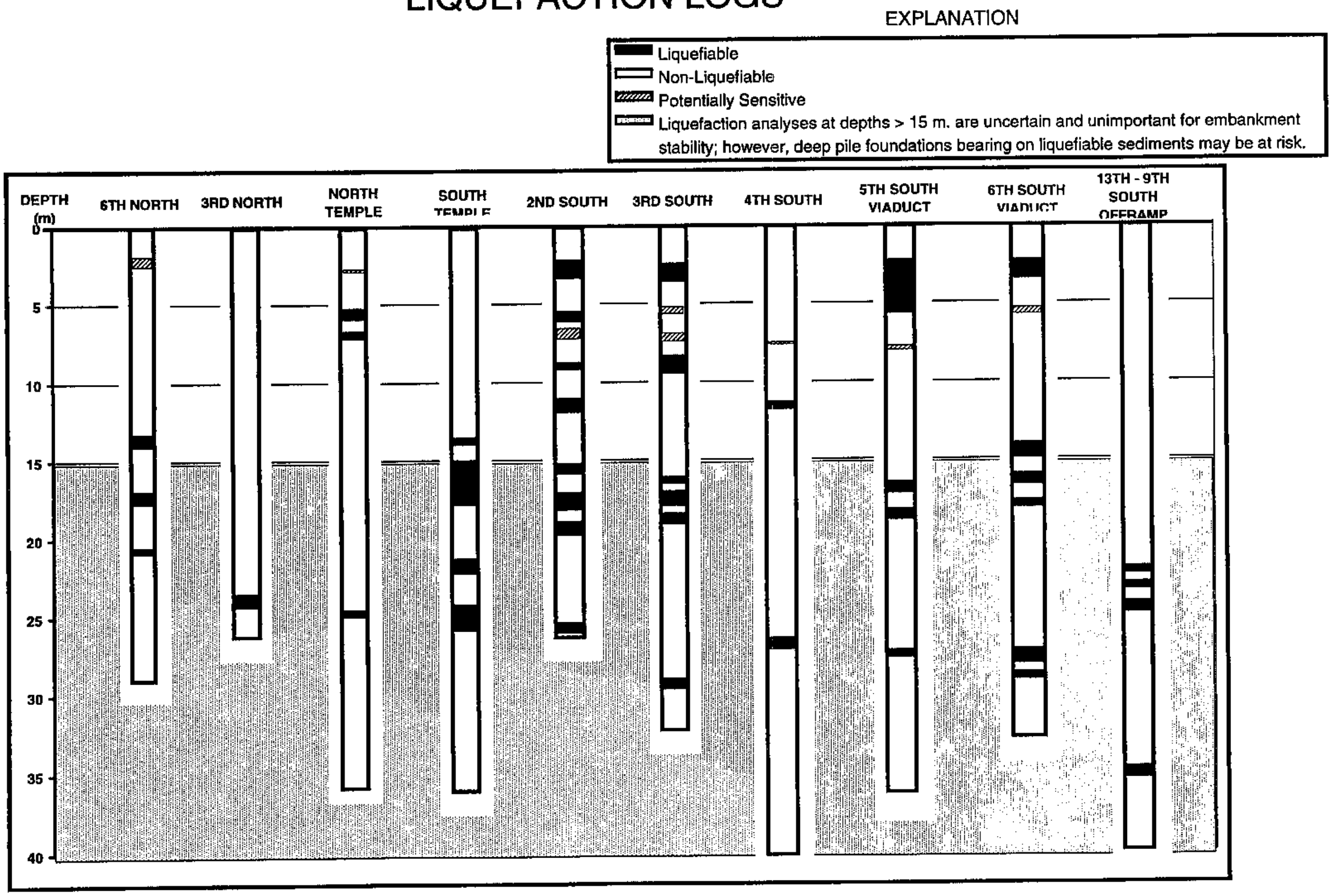


Figure 45: Liquefaction logs for bridge sections between 6th North and the 6th South Viaduct.

These layers vary from low to high certainty of liquefiable material based on calculated I_c values. Bridge sites at 4th West and 8th South are also underlain by approximately 2 m of potentially sensitive clay in the upper 10 m. These thick liquefiable or sensitive layers may lead to embankment instability or perhaps lateral spread. Scattered liquefiable layers lie below 15 m at each site, indicating a minor hazard to bearing capacities of deep foundations. The 13th South site has a 6 m thick layer of potentially sensitive clay below 17 m, which could adversely affect the bearing strength of pile foundations.

I - 80 Interchange and 27th South (figure 46)

The I - 80 interchange and 27th South bridge sites have the greatest thickness of liquefiable sediments in the corridor. Both sites are underlain by 5 m to 7 m of liquefiable soil in the upper 10 m. These liquefiable layers have a low to moderate certainty of liquefaction as shown in figure 46. Further laboratory soil testing is needed to confirm whether this layer is indeed liquefiable. Embankment stability, lateral spread, and ground settlement are possible hazards to structures at these sites. Approximately 5 m to 10 m of collective liquefiable sediments were detected between 15 m and 26 m, which may adversely affect the load capacity of pile foundations. The previous liquefaction analysis that relied on incomplete and poor quality data did not detect liquefiable sediments in the upper 15 m at the I-80 interchange. Using the more complete data for the integrated analysis approximately 5 meters of potentially liquefiable soil was detected in the upper 10 m at the I-80 interchange. This possibly liquefiable layer is one of the more significant layers because of the thickness and the importance of the bridge.

33rd South to D & RGW RR (bridge site at approx. 43rd South) (figure 47)

This segment is underlain by thin sporadic liquefiable layers, 0 m to 2 m thick, between the depths of 12 m and 15 m. The 39th South Bridge site contains 6 m of liquefiable sediment between 18 m and 26 m, which may adversely affect the bearing capacity of deep pile foundations.

45th South to 59th South (figure 47)

Liquefiable layers, 2 m to 6 m thick, at depths between 4 m and 12 m lie beneath each bridge site between 45th and 59th South. These layers vary from a low to high certainty of liquefiability. These layers could affect embankment stability and lead to lateral spread and ground settlement. Deeper liquefiable layers also occur beneath this segment, including the Vine street overpass with 6 m of liquefiable soil between 19 m and 27 m. That layer could adversely affect bearing capacities of deep foundations.

I - 215 Interchange to 146th South (figures 47 and 48).

No CPT data, excluding 90th South, was available for this section nor was SPT data available for the bridges at 106th, 114th, 123rd, or 146th South. The latter four sites were not analyzed using the integrated analysis. The results of the simplified procedure used in the previous analysis are found in section 6.9.3 and figure 44. The SPT liquefaction analyses

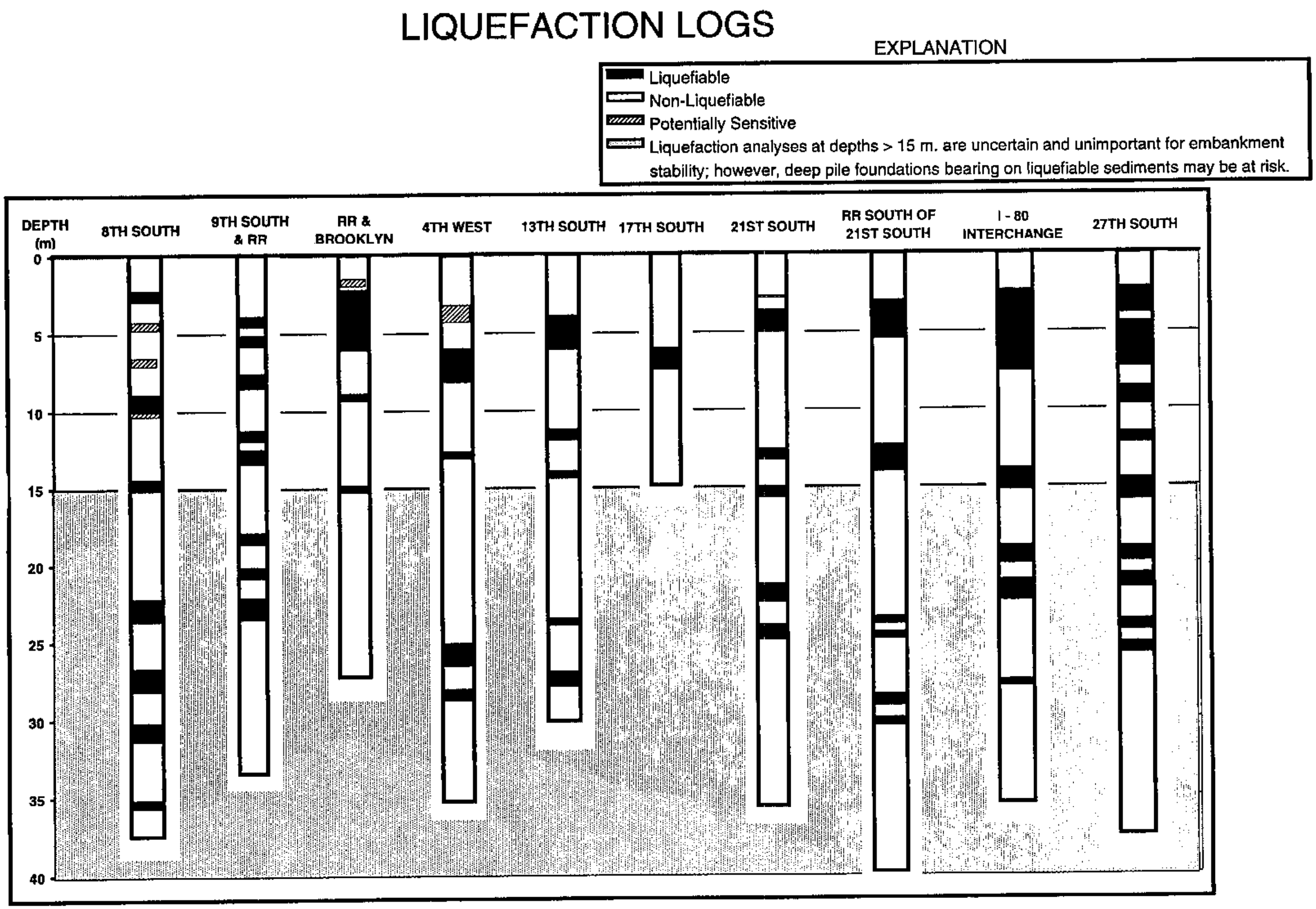


Figure 46: Liquefaction logs of bridge sections between 8th South and 27th South.

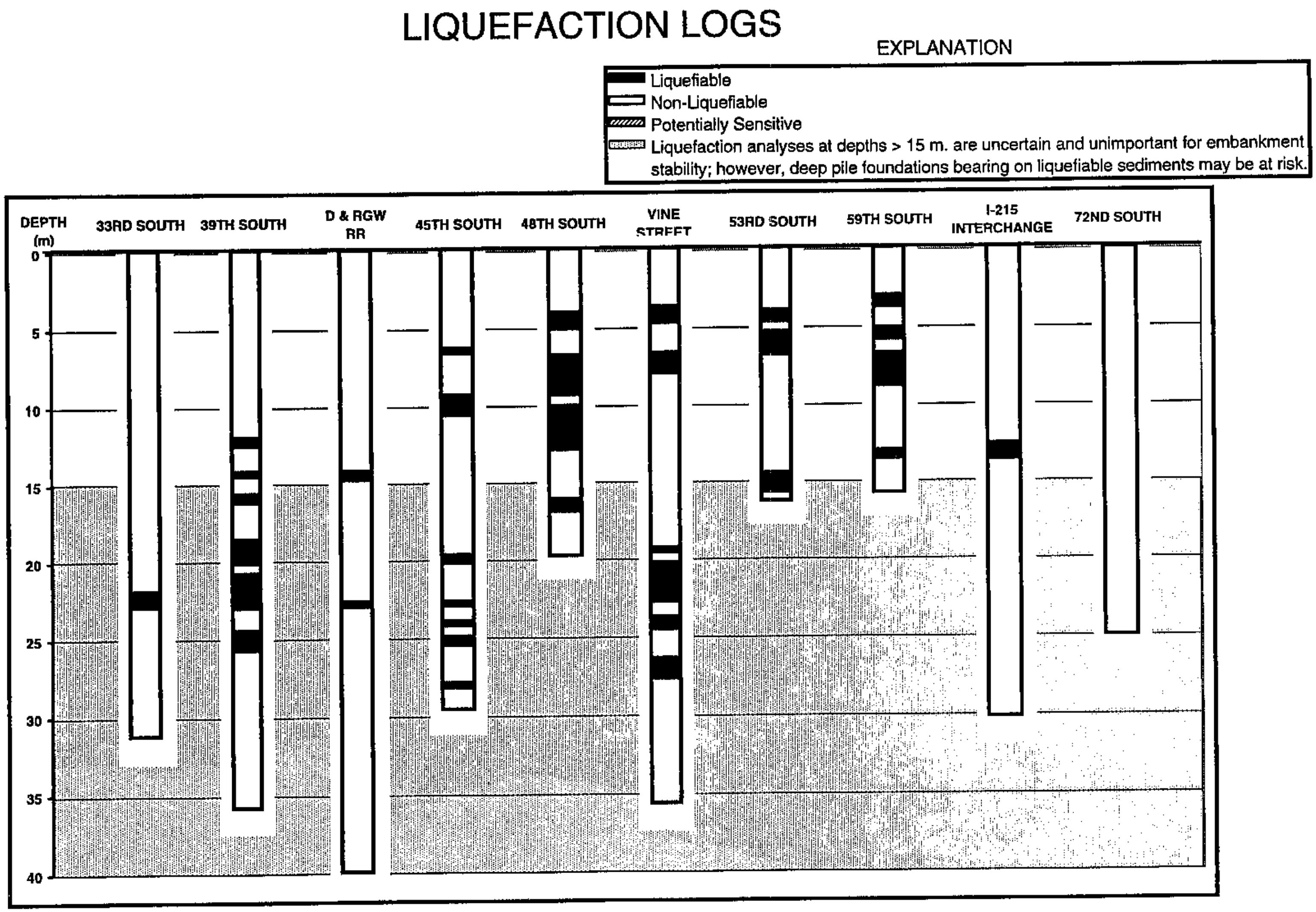


Figure 47: Liquefaction logs for bridge sections between 33rd South and 72nd South.

revealed no liquefiable sediments beneath bridge sites at 72nd South and Center Street in Midvale. The remaining bridge sites are underlain by 2 m to 3 m thick layers of potentially liquefiable soils. At 72nd South, the water table is below 15 m, which effectively eliminates liquefaction hazard to structures at this location.

6.11 I-215 (Belt Route) in Salt Lake County

This section of interstate lies almost entirely in sections of the Salt Lake Valley which are classified by Anderson and others (1994d) as moderate, high or very high liquefaction potential. The only bridge sites located in zones of low potential are at 2000 East and from 6200 South to I-80 on the east side (including 6200 South, 4500 South, 3900 South, 3300 South, and I-80 south-bound off-ramp).

Borehole data was available for all of the bridge sites with the exception of I-215 northbound lane over I-15 and the bridge over 700 west. These sites were classed as Priority III because they are located in areas zoned as high or moderate liquefaction potential by Anderson and others (1994d). Of the 42 bridge sites, 32 were analyzed using the simplified procedure. Of the 32 sites, 2 are assigned Priority I, (both the north and south crossing of I-215 over the Jordan River) and 30 are Priority II (high priority for further investigation). Two bridge sites are Priority III (insufficient information) and eight are Priority IV (low liquefaction hazard). Figures 49 to 51 delineate the liquefiable layers beneath the Priority I and II bridge sites.

6.12 I-15 in Utah County

From Salt Lake County in the north to Juab County in the south, I-15 in Utah County crosses liquefaction zones ranging from low to very high potential (Anderson et. al., 1994c). In all, there are 55 bridges along I-15 in Utah County. Using prior liquefaction analyses by Mabey and Youd (1989) and Anderson and others (1994c), 11 of these sites, Alpine exit, RR north of Lehi, 8800 West, 300 West (Lehi), 200 East (Lehi), 600 East (Lehi), 800 North (Orem), 400 North (Orem), and the three sites in Santaquin, were identified as having little or no liquefaction potential and assigned Priority IV.

The data quality from Main St. in Lehi down to approximately 950 South in Provo was apparently poor. Although the sites were investigated with standard equipment, the corrected blow counts, $(N_1)_{60}$, at most sites were found to be much higher than comparable holes tested with standard equipment a few years later. Comparison with the newer higher quality data indicated that using a lower hammer energy ratio, ER, of 40 percent with the earlier data gave more compatible results. Table 13 shows the comparison of $(N_1)_{60}$ for three bridge sites near 900 North in Provo. The table compares two adjacent holes (one of older data and one of newer data) at each bridge. The older data uses a lower hammer energy ratio of 40 percent and the newer data uses ER of 50 percent. Table 13 shows that using ER=40 percent with the older data brings it more into the range of the newer data which uses ER=50 percent. Based on the comparison of these three holes, all of the analyses using the older data from the bridge sites from Main St. in

LIQUEFACTION LOGS

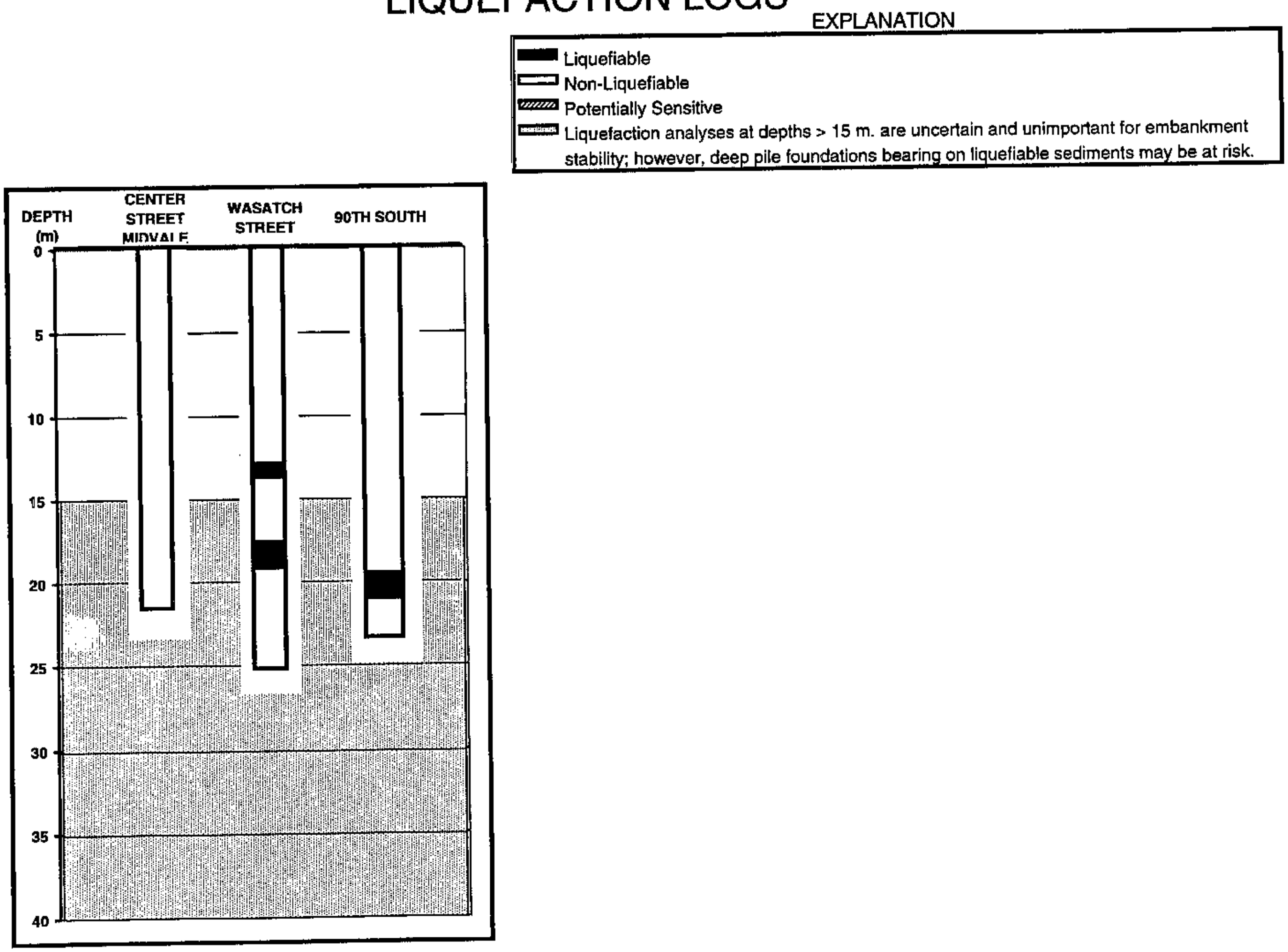
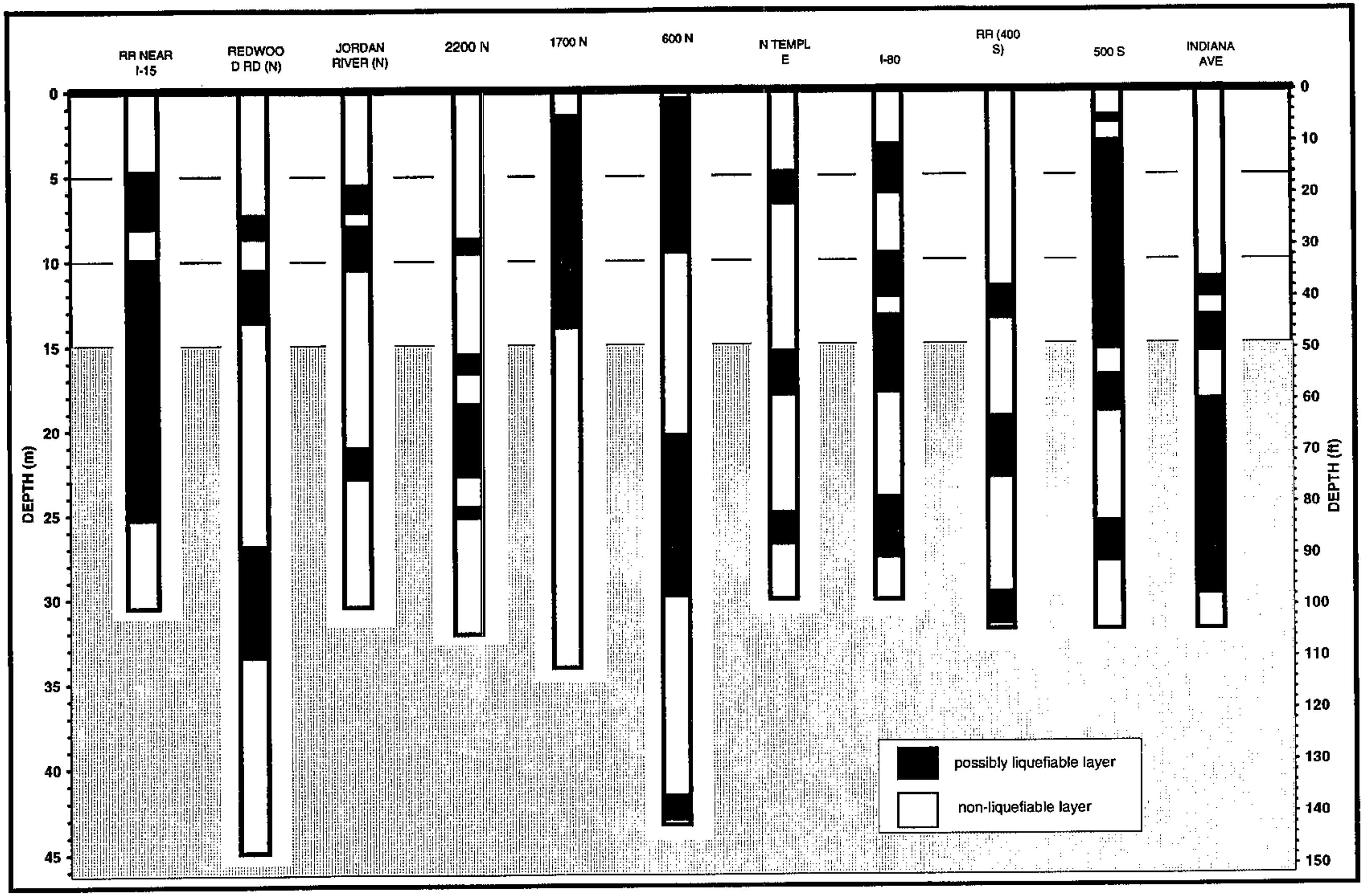
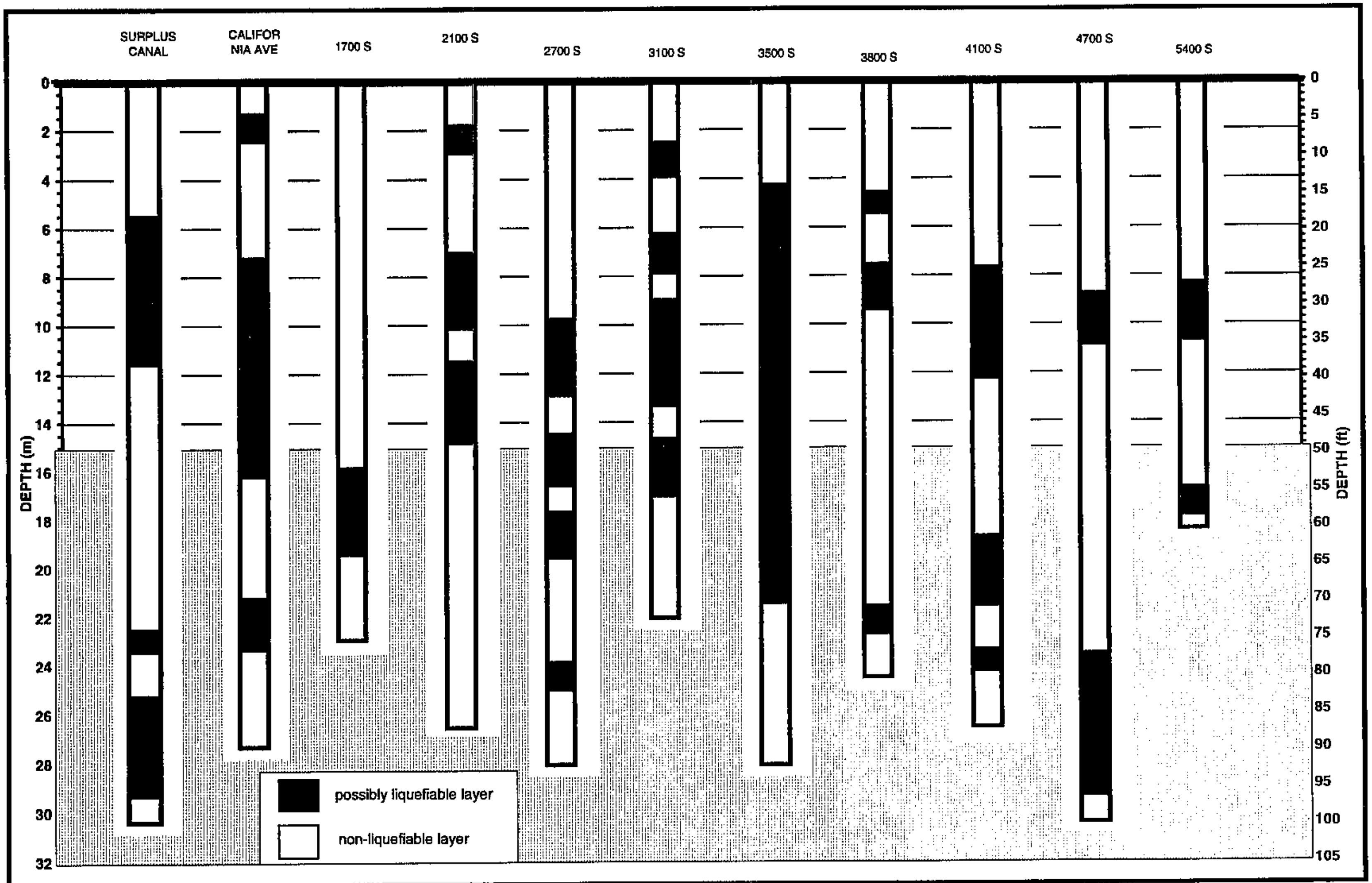


Figure 48: Liquefaction logs for bridge sections between Center Street Midvale and 90th South.



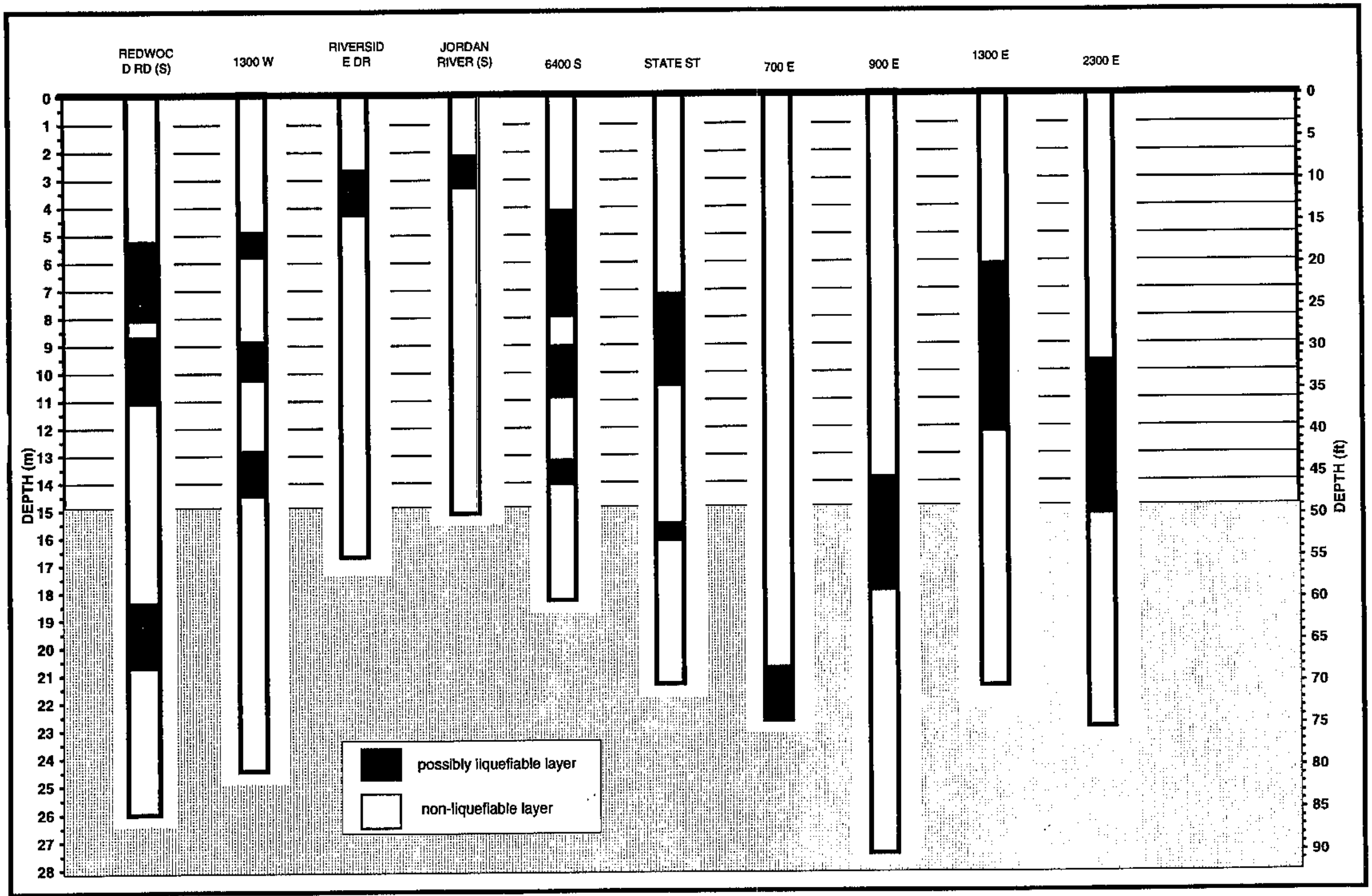
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 49: Logs for I-215 from I-15 (north) to Indiana Ave showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 50: Logs for I-215 from California Ave to 5400 South showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 51: Logs of I-215 from Redwood Rd. to 2300 East showing potentially liquefiable layers in soil profile.

Table 13: Comparison of $(N_1)_{60}$ for three bridge sites near 900 North in Provo comparing two adjacent holes (one of older data and one of newer data) at each bridge. The older data uses a lower hammer energy ratio of 40 % and the newer data uses ER of 50 %. The table shows that using ER=40 % with the older data reduces it into the range of the newer data which uses ER=50 %.

I-15 OVER UPRR			
DEPTH OF SAMPLE		N160	
		OLDER DATA ER=40% HOLE 3K3	NEWER DATA ER=50% HOLE 3
(m)	(ft)		
-1.52	-5	16	9
-3.05	-10	12	11
-4.57	-15	5	4
-6.1	-20	4	5
-7.62	-25	7	4
-9.15	-30	12	11
-12.2	-40	18	6
-18.29	-60	35	39

BRIDGES OVER RG&WRR AND UPRR			
DEPTH OF SAMPLE		N160	
		OLDER DATA ER=40% HOLE 157a	NEWER DATA ER=50% HOLE 5
(m)	(ft)		
-1.52	-5	9	8
-3.05	-10	6	9
-4.57	-15	3	3
-9.15	-30	18	18
-10.67	-35	10	17
-12.2	-40	13	15
-18.29	-60	31	33

900N			
DEPTH OF SAMPLE		N160	
		OLDER DATA ER=40% HOLE 3L4	NEWER DATA ER=50% HOLE 1
(m)	(ft)		
0	0	6	8
-4.57	-15	6	7
-6.1	-20	4	4
-7.62	-25	1	3
-12.2	-40	13	13

Lehi down to 950 South in Provo used ER=40 percent instead of 50 percent. Some sites along I-15 were tested with non-standard equipment. The nine sites, UPRR north of Spanish Fork, RR north of Spanish Fork, Main St., 300 West, 400 North, Spanish Fork Spur, Leland-Benjamin Rd., Spanish Fork River, Arrowhead Rd., and Leland Benjamin Rd., are prefixed with a "U" in appendix A. SPT blow counts were estimated using figure 30.

Of the 42 bridge sites evaluated using site-specific data, all but 3 (Main St. in Lehi, 2000 West Lindon, and Provo River) are underlain by possibly liquefiable layers. The subsurface at the Provo River Bridge consists primarily of dense gravels whereas the other two bridge sites are underlain by clays.

Although the site-specific analyses show potentially liquefiable layers beneath the bridges, they appear to be interconnected only locally without any pervasive liquefiable layer throughout. Figures 52 to 55 delineate liquefiable layers beneath the bridge sites in Utah County. Of the 55 bridge sites in Utah County, 2 are river crossings but only the Spanish Fork River bridge site is Priority I. 39 are Priority II, 1 is Priority III, and 14 are classed as Priority IV.

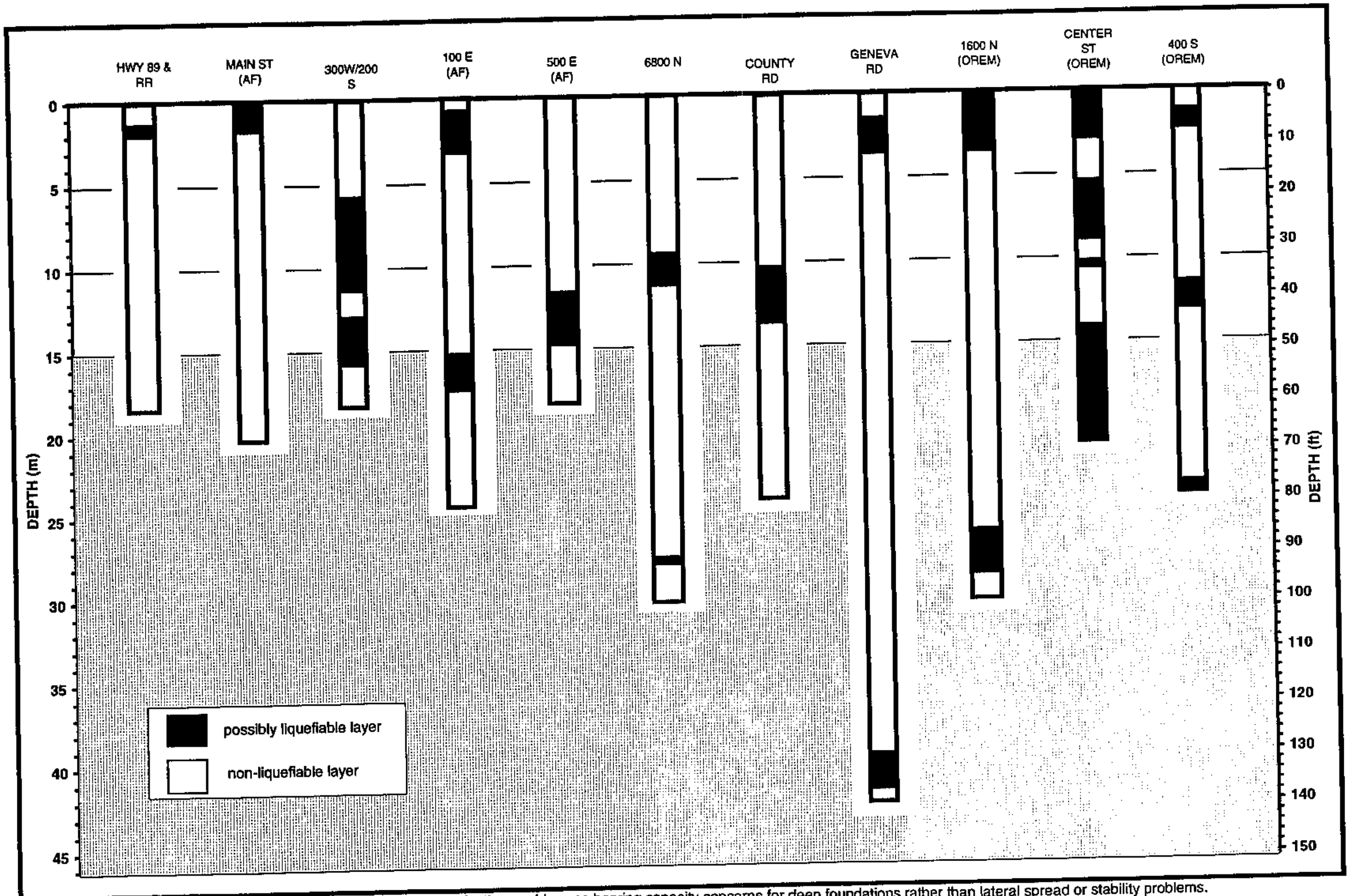
6.13 I-15 Areas South of Utah County

The only segments of I-15 south of Utah County shown by Anderson and others (1994e) as traversing areas of moderate or high liquefaction potential are south of Levan, near Chicken Creek Reservoir, and the area near Sevier Bridge Reservoir and the Sevier River. Four bridge sites are located in these areas. The West Levan Interchange, UPRR, Yuba State Park Interchange, and the Sevier River bridge all are underlain by potentially liquefiable layers as shown in figure 56.

The remaining 14 bridge sites listed in appendix A, Frontage Rd "A", "X, I, and L" lines, "A, L, J, and K" lines in Beaver, Beaver River, "G" line, Paragonah interchange, "G" line, South Parowan interchange, and "B" line, were analyzed because they are located in areas zoned by Hecker and others (1988) as having a shallow water table. The analysis using site-specific data revealed that only four of the thirteen bridge sites south of the Sevier River are underlain by potentially liquefiable layers. Figure 56 delineates the liquefiable sediment beneath the four bridges ("L" line, "G" line, South Parowan interchange, and "B" line). Of the many bridge sites from Utah County to Washington County, only one (Sevier River bridge) was Priority I. Seven sites were classed as Priority II with the remaining sites as Priority IV.

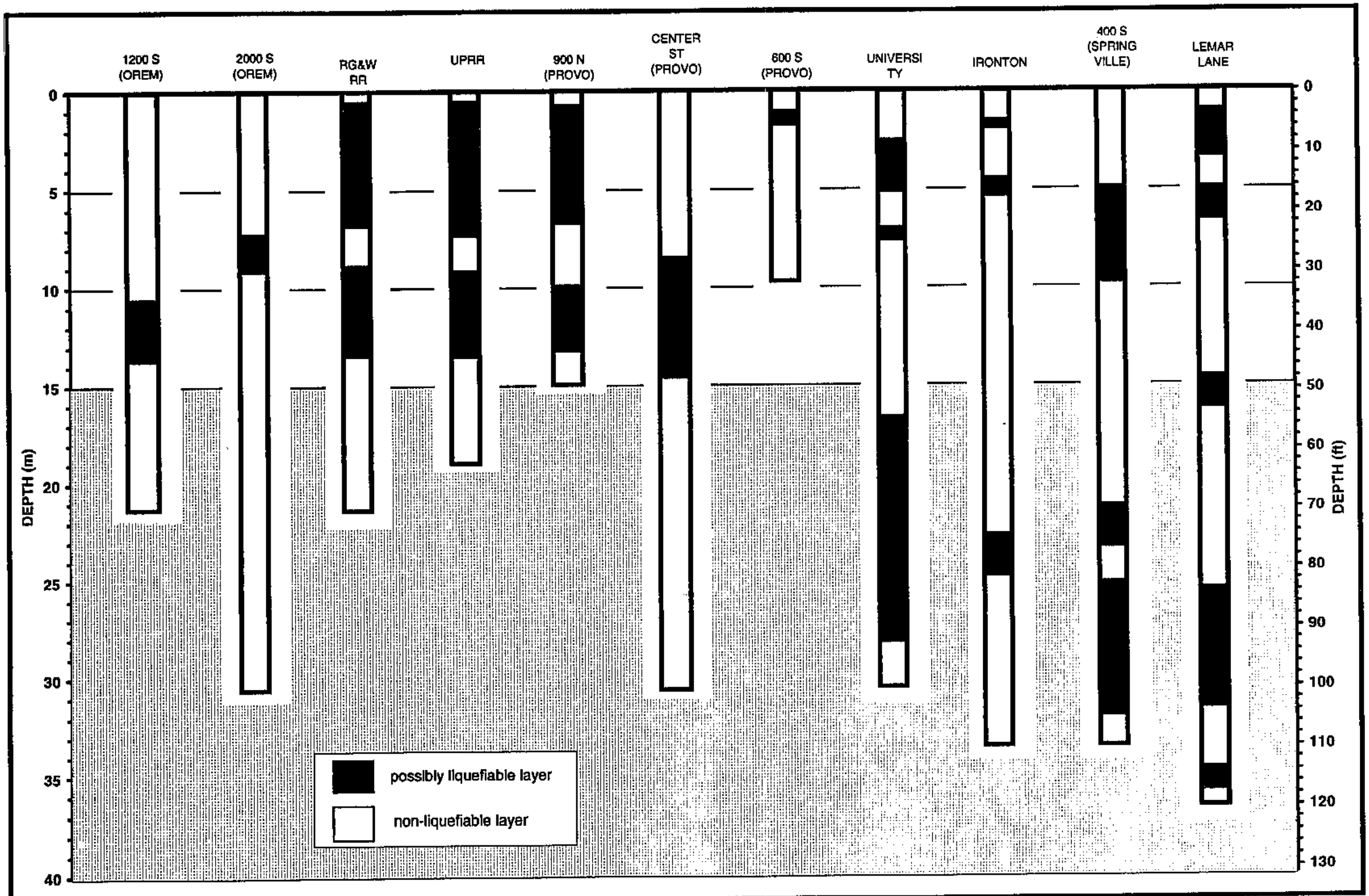
6.14 I-70 from I-15 to Colorado Border

I-70 runs from the junction with I-15 to the Colorado Border. Mabey and Youd (1989) show that all of the interstate east of the 111° meridian has a low liquefaction hazard based on the low predicted Liquefaction Severity Index (LSI). West of the 111° meridian the interstate was screened using the shallow groundwater map by Hecker and others (1988) and the liquefaction hazard maps by Anderson and others (1994e). They show that the only sites with



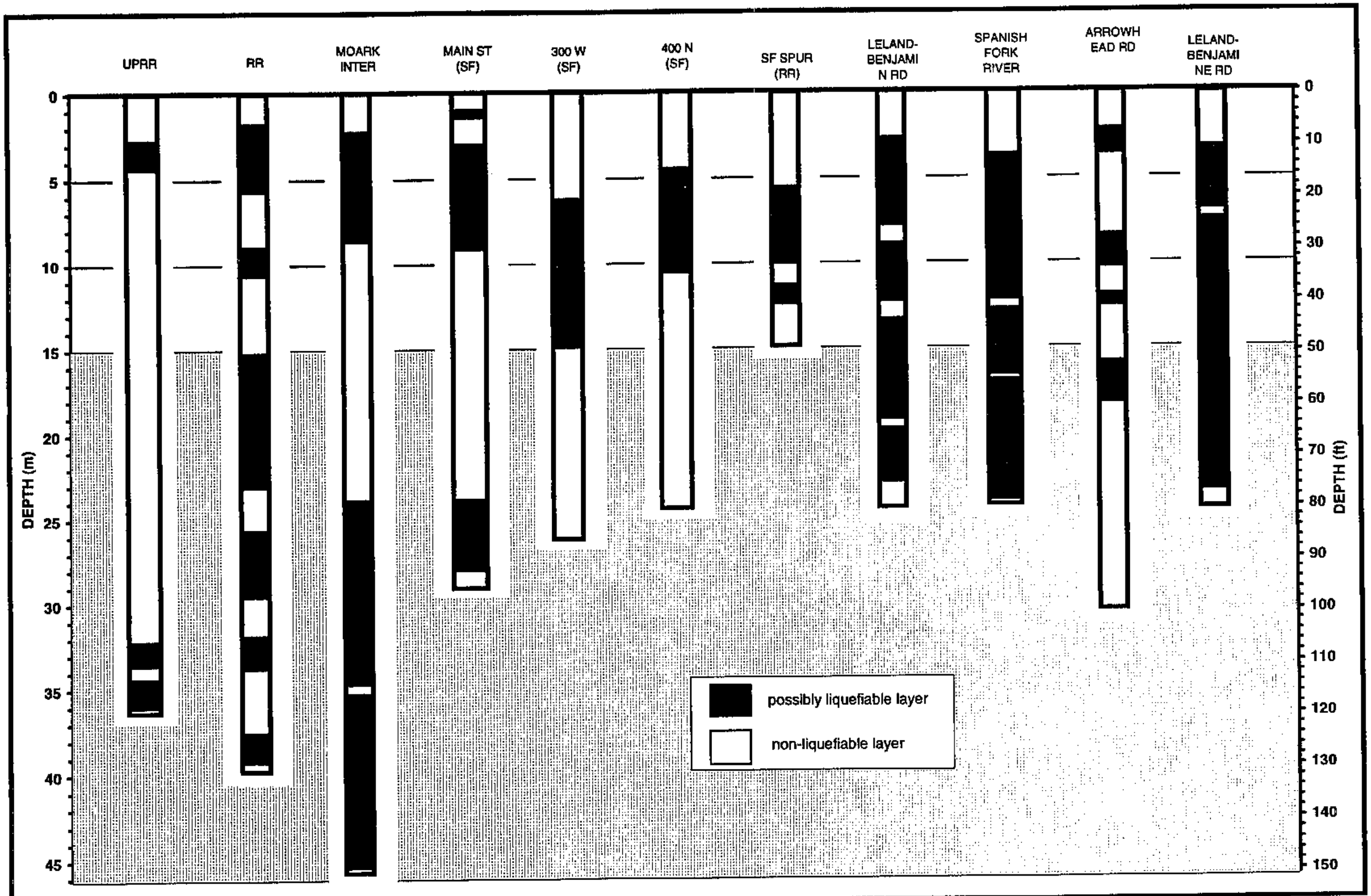
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 52: Logs for I-15 in Utah County from Lehi to Orem showing potentially liquefiable layers in soil profile.



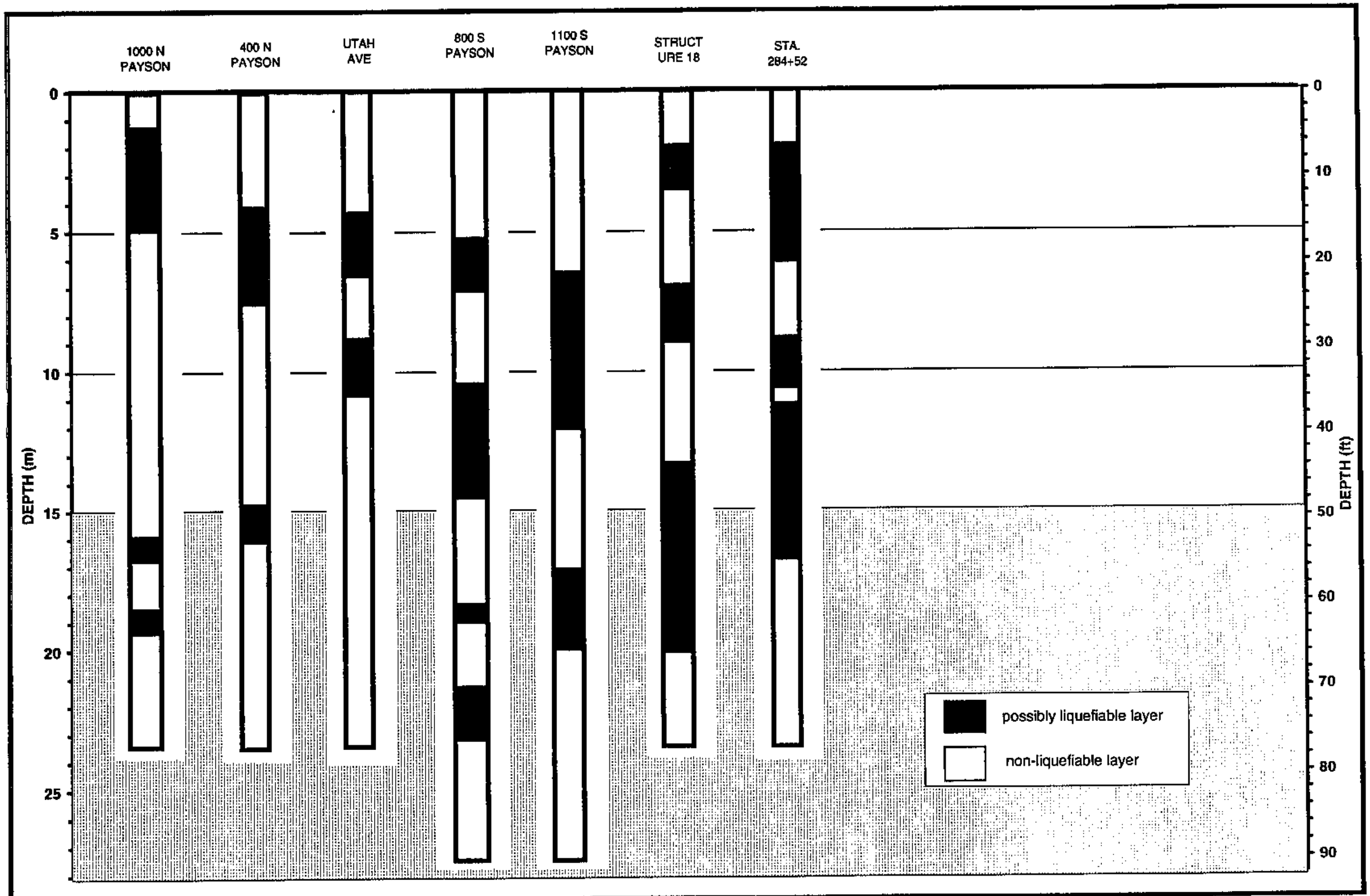
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 53: Logs for I-15 in Utah County from Orem to Springville showing potentially liquefiable layers in soil profile.



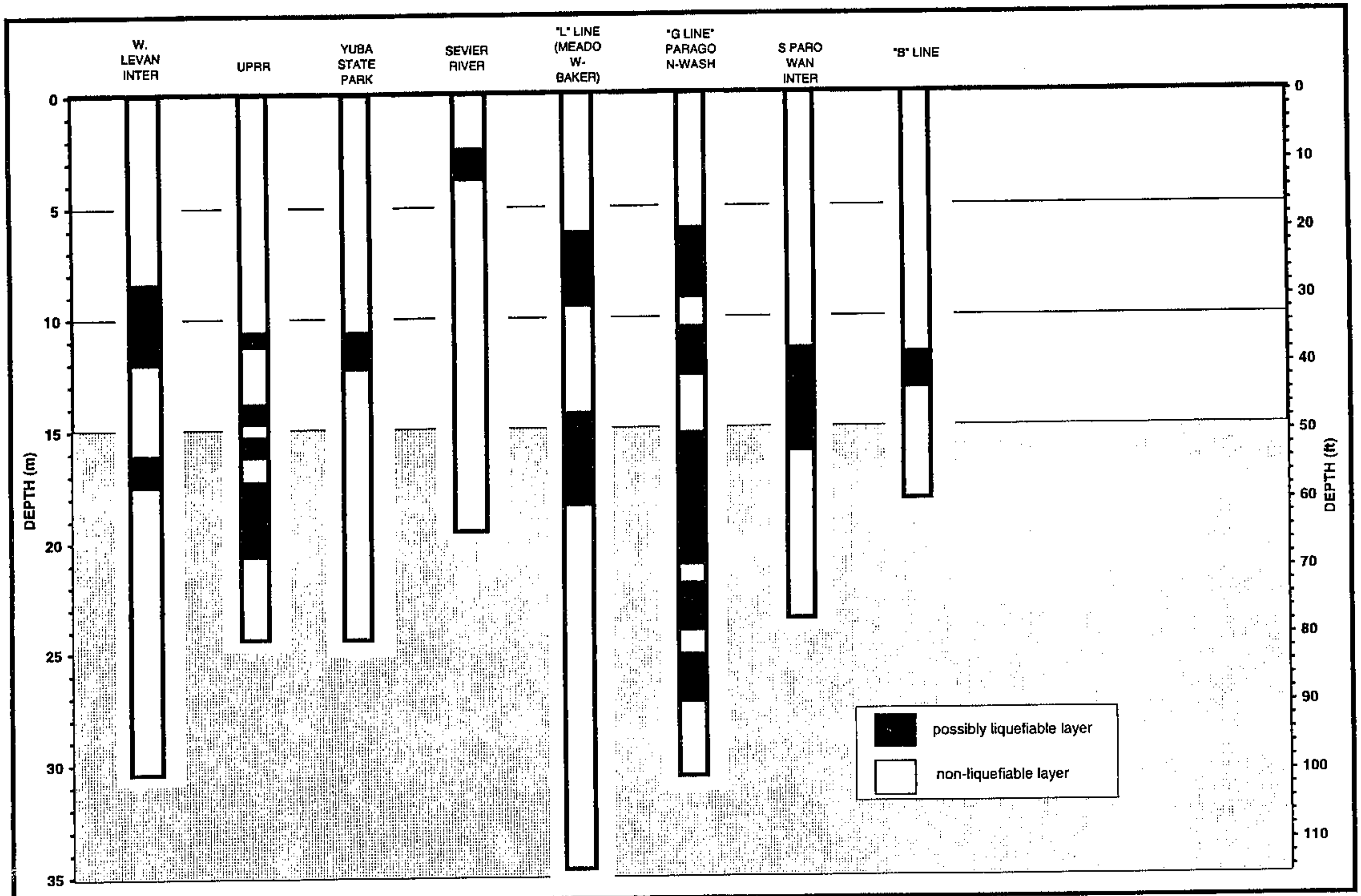
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 54: Logs for I-15 in Utah County from Springville to Payson showing potentially liquefiable layers in soil profile.



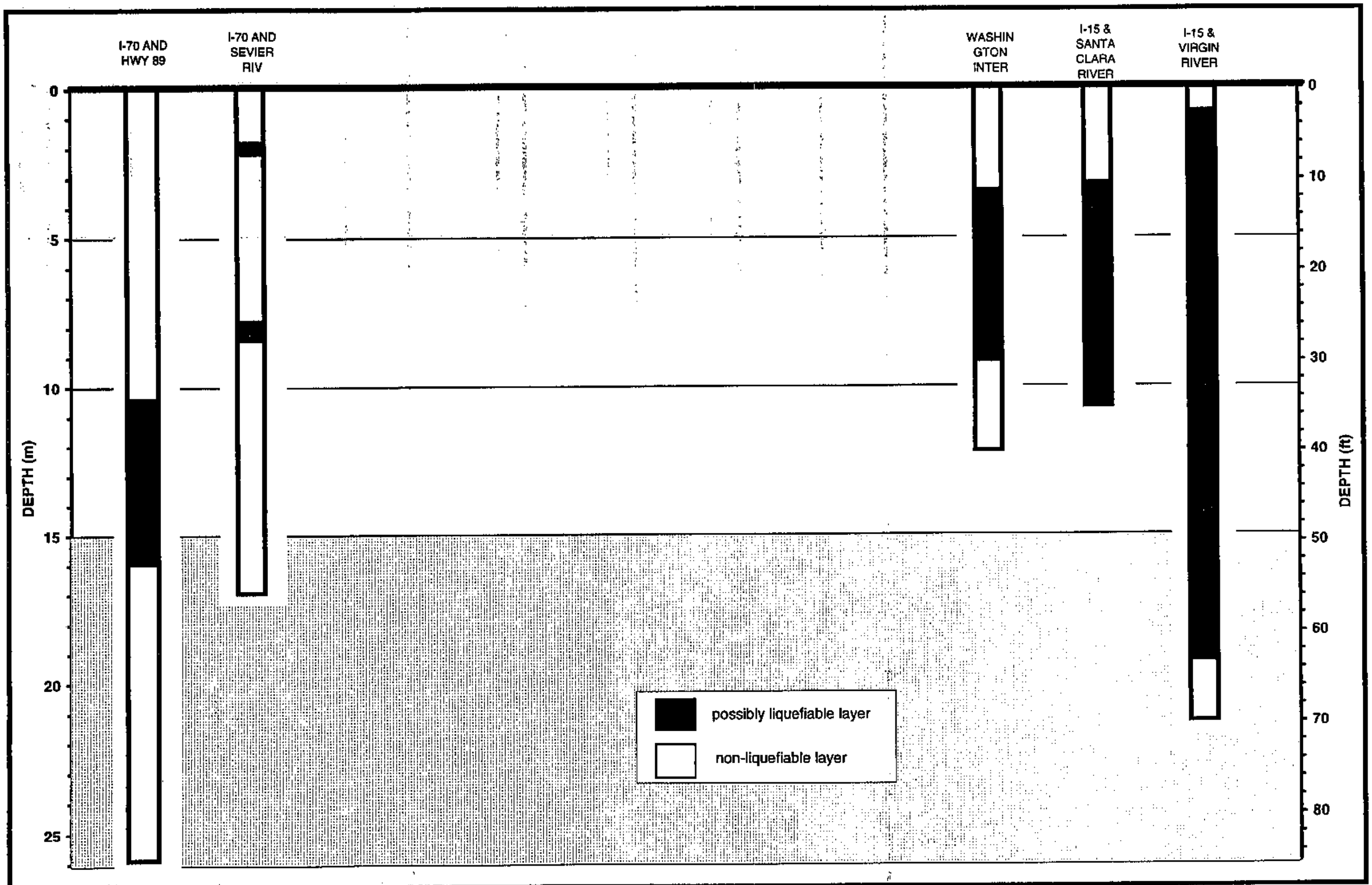
Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems. Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 55: Logs for I-15 in Utah County from Payson to Santaquin showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 56: Logs for I-15 from Utah County to Washington County showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 57: Logs for I-70 and I-15 near St. George showing potentially liquefiable layers in soil profile.

potential liquefaction hazard are located along the Sevier River floodplain near Salina. Of the estimated 59 bridge sites along I-70, only 4 were evaluated using site-specific criteria. Two bridge sites, at Green River and South Salt Wash, were analyzed using site-specific data to validate the effectiveness of the LSI, shallow groundwater maps, and liquefaction hazard maps. Neither site has potentially liquefiable layers. Figure 57 shows the possibly liquefiable layers beneath the other two bridge sites, I-70 over the Sevier River and over Hwy. 89. The Sevier River Bridge is Priority I with high priority for future investigation. The Highway 89 bridge is Priority II, high priority for future investigation. The approximately 57 remaining bridge sites on I-70 are Priority IV, low liquefaction potential.

6.15 I-15 in St. George Area

This section of interstate runs from the Washington County line down to the Arizona border. Liquefaction potential maps have not been developed for the region so other reports were used for the regional screen. Black and others (1992) reported cases of liquefaction and lateral spread along the Virgin River after the 1992 Washington County earthquake. Hecker and others (1988) show that the region along the Virgin River and around St. George has a shallow groundwater table (less than 9 m deep). Based on these two reports, six sites were evaluated using site-specific criteria.

The Virgin River and Santa Clara River bridge sites were the only bridge sites not investigated using standard equipment. SPT blow counts for these sites, those boreholes with the prefix "U" in appendix A, were estimated using figure 30 as explained in section 6.3.

The simplified procedure revealed that only three sites in the area have sediments susceptible to liquefaction. These sites, Washington interchange and I-15 over the Virgin and Santa Clara Rivers, are shown in figure 57. Of the estimated seven bridge sites in the area, two, the Virgin River and Santa Clara River bridges, are Priority I with a high priority for future investigation. The Washington interchange is Priority II and the remaining four sites are classed as Priority IV.

6.16 Federal and State Highways

The most significant problem encountered in searching for data from the State roads as well as Federal highways was the lack of data for structures built prior to the 1950's or 1960's. Unless the bridge had been rebuilt, expanded or replaced in the last few decades, the foundation investigations for the structures were either lost or nonexistent. The Federal highways reviewed included Highways 89, 91, 189, 40, 6, 50, 191, 163, and 666. Two State roads, SR 30 between Logan and Tremonton and SR 201 (2100 South Freeway) were also investigated.

6.16.1 Hwys. 191, 666, 163

Mabey and Youd (1989) show that all of the State east of the 111° meridian has a low liquefaction hazard based on the low values of Liquefaction Severity Index. The entire lengths of Highways 191, 666, and 163 lie east of the 111° meridian so the few bridge sites on these roads were given low liquefaction hazard and low priority for further study (Priority IV).

6.16.2 Hwys. 50, 6

Based on the LSI maps (Mabey and Youd, 1989) as well as the liquefaction potential maps by Anderson and others (1994e), Highway 50 does not cross any potentially hazardous zones along its entire length (from Nevada border to Colorado border). However, Mabey and Youd (1989), and Anderson and others (1994c,e), show that stretches of Highway 6 could have a potential liquefaction hazard. The potentially hazardous areas run from Goshen to about 2.5 miles east of Goshen, and from Payson to just east of Spanish Fork. However, road maps of these areas show that the only location with a major bridge structure is at the Highway 6 crossing over the Spanish Fork River in Utah County. No borehole logs could be found for that site so based on the liquefaction hazard maps by Anderson and others (1994c,e) and because the site is located at a river crossing, it is classed as Priority I, a potential hazard and high priority for further investigation.

6.16.3 Hwy. 40

From Silver Creek Junction west to the Nevada border, Highway 40 follows Interstate 80 and in many places was replaced by the Interstate Highway. No borehole data could be found for any bridge sites along this western stretch of Highway 40. Because that highway follows I-80 so closely, it was assumed to have similar liquefaction characteristics. All the bridge sites along Highway 40 from I-80 at Silver Creek Junction east to the Colorado border are classed as Priority IV based on liquefaction potential maps, LSI maps, and shallow groundwater maps (Anderson et. al., 1994e; Mabey and Youd, 1989; Hecker et. al., 1988). All the bridge sites west of Silver Creek Junction are Priority III.

6.16.4 Hwy. 189

Using LSI maps (Mabey and Youd, 1989), liquefaction potential maps (Anderson et. al., 1994c), and shallow groundwater maps (Hecker et. al., 1988), Highway 189 is classed as low potential (Priority IV) for liquefaction from Silver Creek Junction at I-80 southward to the mouth of Provo Canyon. Southward from that point, Anderson and others (1994c) show a zone of moderate potential from the mouth of the canyon to the point the highway ends (at 300 South in Provo). Highway 189 does not cross any bridges once it leaves Provo Canyon so there is no significant liquefaction hazard to bridge structures in that area.

6.16.5 Hwys. 89, 91

Highways 89 and 91 have a greater potential for liquefaction hazard than the other Federal highways because they generally traverse within a few kilometers of alluvial areas and are parallel to major faults in Utah. Mabey and Youd (1989) show that both highways run through zones of high LSI along most of their lengths. Along highways 89 and 91, the only bridge sites that lie on potentially liquefiable sediments are at river crossings. The majority of the secondary roads that intersect these highways are at intersections instead of bridges or interchanges. The potentially hazardous river crossings include Hwy. 89/91 over the Logan, Little Bear, Ogden and Provo Rivers, Highway 89 over the Spanish Fork, San Pitch, and Sevier Rivers, and over Clear and Hog Creeks (Anderson et. al., 1994a,b,c,d,e). Many other creeks flow under these roads but do so in culverts so they were not considered in this study.

Highway 91 separates from Highway 89 at Springville and continues south to Nephi roughly parallel to I-15. From Nephi south, I-15 replaced Highway 91 along many sections. No information could be found for any Hwy. 91 bridges south of Nephi. It is probable that Hwy. 91 crosses some rivers such as the Virgin or Beaver Rivers. Any major river crossing along this route should be considered a liquefaction hazard and Priority I for further study.

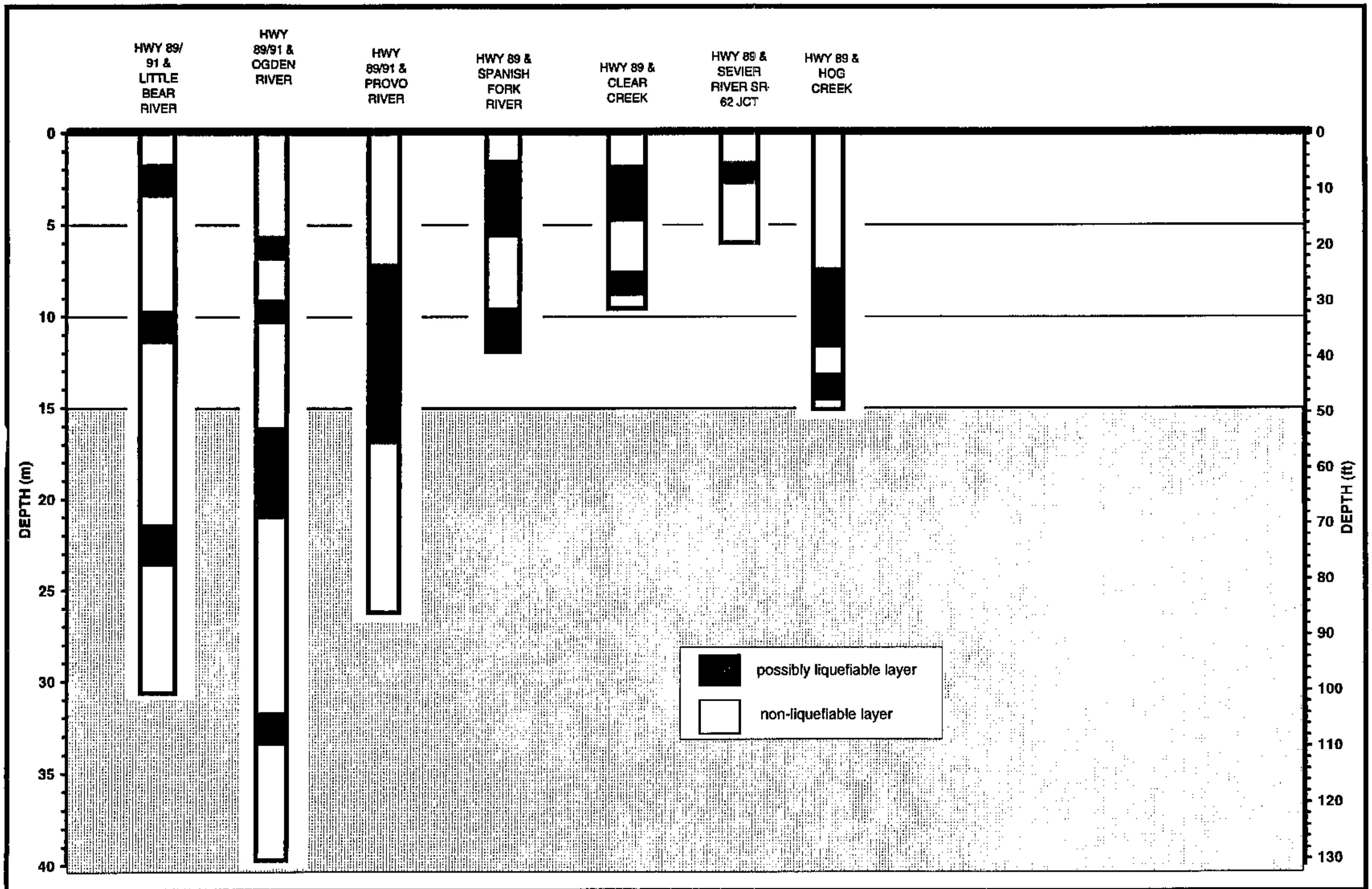
Of the 10 sites along Highway 89 analyzed using site-specific data for this study, seven were found to be underlain by possibly liquefiable sediment. These were classed as Priority I because they are located at river crossings. Three sites were classed as Priority IV. Figure 58 delineates the possibly liquefiable sediment beneath the bridges.

6.16.6 S.R. 30 Between Logan and Tremonton

This stretch of road runs between Logan and Tremonton. Anderson and others (1994a) show high liquefaction potential at the Malad and Little Bear River crossings. Figure 59 delineates the potentially liquefiable layers. The S.R. 30 bridge site over the Malad River has liquefiable soil below 30 m. This depth should not present a problem to embankment stability, but could affect pile load capacity. Both sites are Priority I and should be investigated further to quantify the liquefaction hazard.

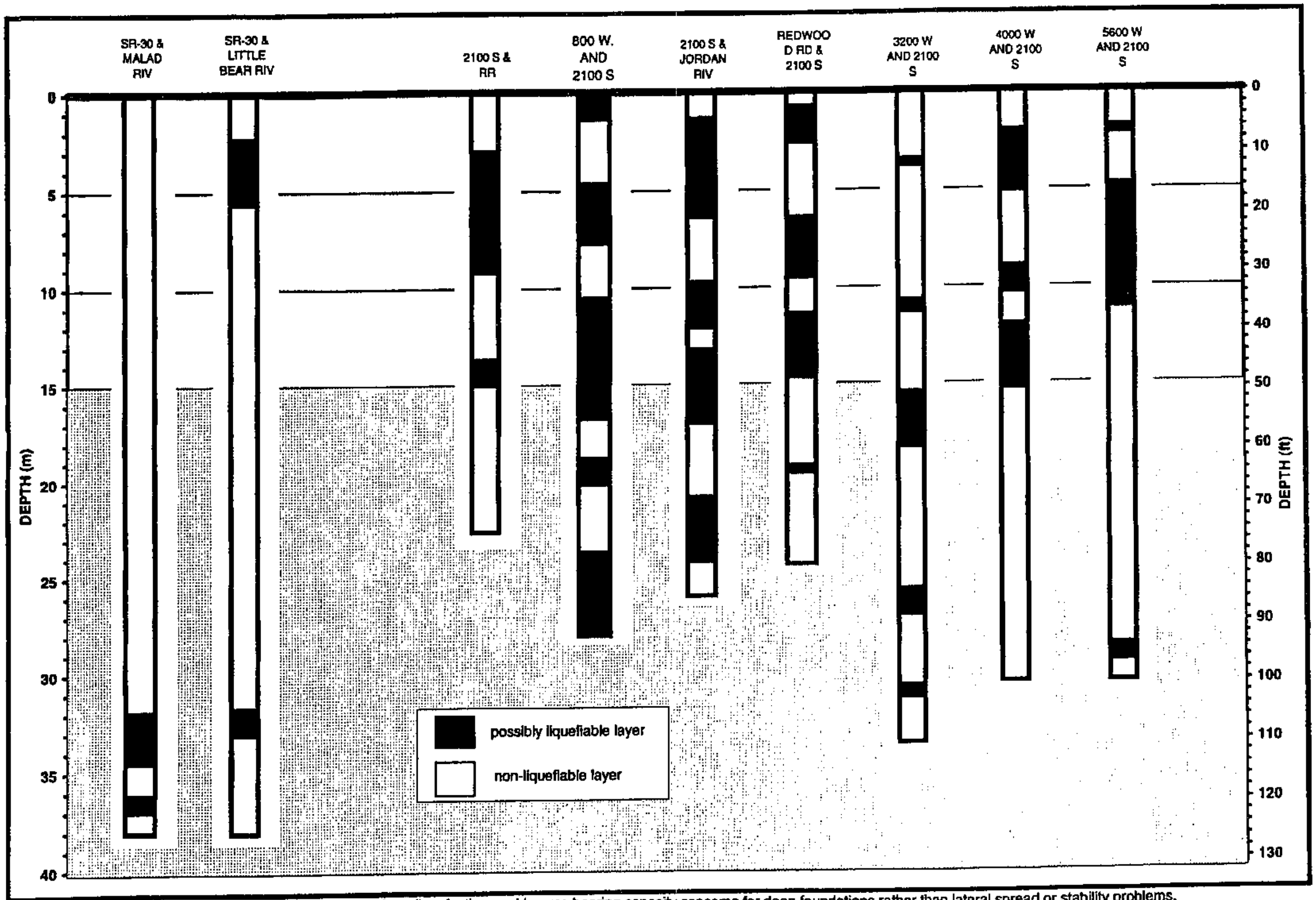
6.16.7 S.R. 201 (2100 South Freeway)

S.R. 201 is a divided freeway which runs west through Salt Lake County from the I-15 junction to the I-80 Blackrock Interchange. The freeway runs parallel to I-80 and passes over similar Lake Bonneville deposits. Foundation investigations could not be found for bridge sites at 2700 West, 3600 West, or any sites west of 5600 West. Because the liquefaction hazard maps (Anderson et. al., 1994d) indicate that the entire highway alignment is underlain by sediments with high liquefaction potential, the estimated five bridges sites in this alignment are considered potentially liquefiable and Priority III for further study. The Jordan River Bridge site is classed as Priority I with high priority for further investigation. The remaining sites at RR, 800 West, Redwood Rd., 3200 West, 4000 West, and 5600 West are all classed as Priority II. Figure 59 shows the liquefiable layers beneath these sites.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems.
Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 58: Logs for Federal Highways 89 and 91 showing potentially liquefiable layers in soil profile.



Note - shaded areas below 15 m (50 ft) delineate layers where liquefaction could cause bearing capacity concerns for deep foundations rather than lateral spread or stability problems. Only sites underlain by potentially liquefiable layers are profiled on this plot. (priority I and II)

Figure 59: Logs for State Roads 30 and 201 showing potentially liquefiable layers in soil profile.

CHAPTER 7 – CONCLUSIONS

The objective of this study was to evaluate the liquefaction hazard at bridge sites throughout the state of Utah. The objective was also to test and suggest revisions to the screening guide developed by Youd (1998). The screening procedure involved a regional screen of liquefaction potential, a site-specific evaluation, and calculation of liquefaction-induced ground failures. The screening guide continues beyond determining liquefaction potential to calculation of the liquefaction-induced ground failures. Due to the limited scope of this study, only an example calculation of ground failures was included.

The final goal of the guide was to prioritize the bridge sites for future investigations. Sites were prioritized according to the presence of liquefiable sediment and the quality and availability of subsurface data. Sites classified as Priority I have a confirmed high hazard of liquefaction or the available information was insufficient to eliminate the possibility of liquefiable sediment. A second criterion is that the site is located in an area highly vulnerable to ground failure, such as river crossings, near lakes or other bodies of water, near a steep slope, or approached by thick embankments (greater than 5 m) overlying soft sediment. Sites classed as Priority II are localities confirmed to be underlain by liquefiable sediments or sensitive clay, but where the available site information is insufficient to fully evaluate ground or foundation stability hazards. These sites are located away from rivers, or other bodies of water, steep slopes, or thick embankments overlying soft sediment. Sites classed as Priority III have insufficient data to investigate them and are located away from river crossings. Liquefaction at these sites could cause ground settlement and possible foundation instability, but damaging lateral ground displacements are unlikely. Priority IV sites are those where the screening evaluation indicated very low liquefaction susceptibility or ground failure.

Approximately 325 bridge sites were reviewed and analyzed using the site-specific steps in the screening guide. The majority of these sites (about 279) were identified as having liquefaction potential (25 Priority I and about 254 Priority II). The 25 Priority I sites have the highest potential hazard and thus the highest priority for future study. While the screening guide proved useful in prioritizing sites for further investigation, the quality and quantity of data proved to be the most significant limiting factor. For many of the bridge sites, subsurface investigations were incomplete or non-existent. Much of the available data was gathered using non-standard procedures and equipment. This general lack of data forced the use of conservative assumptions to prioritize the sites. The results of the liquefaction evaluation of Utah's bridge sites are discussed by freeway segment in the report. The sites analyzed with site-specific information are prioritized in appendix A. The high priority sites (Priority I) which were located at river crossings are listed in table 14. The Priority I sites have the highest likelihood for liquefaction-induced damage due to the high potential of lateral spread of flood plain sediments toward the river channels. Additional prioritization of the Priority I sites should consider the importance of the bridge structure. Therefore, Interstate Highway bridge sites would generally have greater priority than Federal and State highway bridges because of their generally greater importance to the transportation system. Bridges on highways without readily available bypass routes might also be given higher priority based on importance. Table 14 lists the Interstate

Highway river crossings as the highest priority I (1) and the Federal and State highway crossings as priority I (2); slightly lower priority due to the lower importance.

Table 14: Bridge sites located at river crossings with very high priority for further investigation (Priority I). Also listed are the number of boreholes investigated at the site and thickness of possibly liquefiable layers in subsurface.

BRIDGE LOCATION	BORING NUMBER	CUMMULATIVE THICKNESS OF POSSIBLY LIQUEFIABLE LAYERS IN UPPER 15 m (m)	FUTURE STUDY PRIORITY
I-15 OVER MALAD RIVER	6	6.7	I (1)
I-84 / I-15 OVER MALAD RIVER	1	9.7	I (1)
I-15 OVER BEAR RIVER	5	6.4	I (1)
I-15 OVER WEBER RIVER	2	1.2	I (1)
I-84 OVER COTTONWOOD CREEK	2	4.3	I (1)
I-80 OVER JORDAN RIVER	A-3	2.7	I (1)
I-215 OVER JORDAN RIVER (NORTH)	S-17	4.4	I (1)
I-215 OVER JORDAN RIVER (SOUTH)	3	1.1	I (1)
I-15 OVER SPANISH FORK RIVER	U68	10.7	I (1)
I-15 OVER SEVIER RIVER	3	1.2	I (1)
I-70 OVER SEVIER RIVER	6	0.5	I (1)
I-15 OVER SANTA CLARA RIVER	U5	4.7	I (1)
I-15 OVER VIRGIN RIVER	U3	13.8	I (1)
HWY 6 OVER SPANISH FORK RIVER	Insufficient information to assess liquefaction hazard		I (2)
HWY 89/91 OVER LOGAN RIVER	Insufficient information to assess liquefaction hazard		I (2)
HWY 89/ 91 OVER LITTLE BEAR RIVER	DH3	3.4	I (2)
HWY 89/91 OVER OGDEN RIVER	1	2.3	I (2)
HWY 89/91 OVER PROVO RIVER	1	7.8	I (2)
HWY 89 OVER SPANISH FORK RIVER	2	6.1	I (2)
HWY 89 OVER CLEAR CREEK	1	4.1	I (2)
HWY 89 OVER SEVIER RIVER NEAR SR-62 JCT	2	1.0	I (2)
HWY 89 OVER HOG CREEK	1	6.3	I (2)
SR-30 OVER MALAD RIVER	1	0*	I (2)
SR-30 OVER LITTLE BEAR RIVER	2	3.4	I (2)
2100 S (HWY 201) OVER JORDAN RIVER	1	9.7	I (2)

G.W.T. - groundwater table
I (1) - Confirmed presence of liquefiable sediments or insufficient information to assess liquefaction susceptibility with Interstate Highway bridge located at river crossing-- highest priority for further investigation.
I (2) - Confirmed presence of liquefiable sediments or insufficient information to assess liquefaction susceptibility with Federal or State highway bridge located at river crossing—very high priority for further investigation.
* - liquefiable layers are deeper than 15 m

A principal emphasis of this study was on evaluating the liquefaction hazard for the I-15 corridor. The results of the two studies show that nearly all of the I-15 Bridge sites in Salt Lake County are underlain by possibly liquefiable sediment. Most sites through the Salt Lake I-15 corridor have potentially liquefiable sediment in the upper 15 m. The cumulative thicknesses of these liquefiable layers ranged from 1 m to 7 m. These studies also confirmed that higher quality data reduce the uncertainty and conservatism in the simplified procedure. Gilstrap (sections 4.4 and 6.10) shows that using higher quality data and the integrated analysis reduced the amount of material classed as liquefiable by up to 54 percent.

Liquefaction at I-15 bridge sites could lead to embankment instability, lateral spread, and ground settlement which could induce bridge damage or loss of load capacities for deep foundations. An example calculation of liquefaction-induced ground failure hazard was made for the 600 South off-ramp. Standard slope stability analyses indicated that the 9 m high embankments at the site are stable under dynamic conditions with a factor of safety of 1.46. Calculation of embankment deformations indicated small movements of less than 40 mm due to seismic shaking. An analysis of lateral spread potential indicated displacements of less than 150 mm, which are non-damaging to well constructed bridges. The simplified analyses also indicated settlements less than 35 mm, which are generally non-damaging. The result of this hazard analysis is that by assuring that structural loads are transferred downward to competent layers, the liquefaction hazard would be mitigated without adverse consequences to the highway system. Thus, liquefaction hazard can be mitigated at the 600 South off-ramp through structural measures. If predicted embankment deformations or lateral spread displacements had been potentially damaging, other remedial measures, such as soil improvement, may have been necessary.

In summary, large parts of Utah are underlain by sediment with significant liquefaction potential. The majority of the bridge sites in the northern counties (Weber, Davis, Salt Lake, Tooele, and Utah) lie in these high hazard areas and are Priority I or II sites. Priority I and II bridge sites in the rest of the state are limited primarily to river crossings (Logan, Little Bear, Bear, Malad, Sevier, Virgin, and Santa Clara Rivers).

The screening guide proved to be useful for the rapid assessment of liquefaction hazards. It outlines explicit steps which allow for site screening without subsurface investigations. The guide details a simple yet accurate and conservative, repeatable screening process.

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APPENDIX A - RESULTS OF SITE-SPECIFIC EVALUATION OF BRIDGE SITES

Appendix A - Results of site-specific evaluation of bridge sites.

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
I-15 (AND PART OF I-84) IN BOX ELDER COUNTY								
"S" LINE UNDER I-15	3	33.5	7.5	0.5	7.0	G.W.T.-23.16, 26.82-31.09	184.1	II
MALAD RIVER AND I-15	6	29.0	7.5	0.5	1.5	G.W.T.-6.71, 8.23-9.75, 19.2-25.6	207.9	I
"T" LINE (HIGH SCHOOL RD?)	4	18.3	7.5	0.5	0.7	3.96-8.84	67.1	II
MAIN ST. TREMONTON	4	42.7	7.5	0.5	1.8	9.14-10.36	30.5	II
"A" AND "D" LINES OVER I-84	3	39.6	7.5	0.5	2.1	6.1-10.06	103.6	II
THACHER SPUR UPRR	3	35.1	7.5	0.5	0.6	3.35-10.06, 24.99-31.39	136.2	II
"A" LINE OVER IOWA STRING RD.	1	45.7	7.5	0.5	0.9	5.49-6.71	30.5	II
IOWA STRING RD.	3	48.8	7.5	0.5	0.5	2.74-7.62, 13.11-16.46, 25.3-30.48	87.8	II
UPRR AND MALAD RD.	2	33.5	7.5	0.5	0.3	0.91-5.49, 6.1-18.29, 28.04-29.87	265.2	II
COUNTY RD. (STA 2495+63)	1	45.7	7.5	0.5	0.6	G.W.T.-7.62, 24.69-44.81	100.0	II
I-84 / I-15 OVER MALAD RIVER	1	51.8	7.5	0.5	0.0	G.W.T.-3.35, 7.62-14.02, 15.54-17.98, 19.51-26.21, 36.27-37.8,	304.8	I
U.S. 30S/191 OVER I-84/I-15	3	44.2	7.5	0.5	2.4	5.79-19.81, 27.43-37.49	165.8	II
"O" LINE OVER "N" LINE (NEAR ELWOOD INTERCHANGE)	4	33.5	7.5	0.5	3.1	8.53-21.03, 28.04-29.41	96.0	II
BEAR RIVER	5	16.8	7.5	0.5	0.3	2.44-7.32, 12.5-14.02, 15.24-16.76	70.1	I
BRIDGE AT STA 2373+28	U05	25.9	7.5	0.5	1.2	13.72-16.76, 17.98-24.84	#	II
HONEYVILLE INTERCHANGE	U04	24.4	7.5	0.5	1.5	G.W.T.-4.57, 11.89-14.33, 15.85-16.76, 19.2-24.38	#	II
CALLS FORT ROAD	U03	30.5	7.5	0.5	1.5	G.W.T.-2.44, 14.33-17.98, 21.03-24.08, 26.52-28.96	#	II
RD. NEAR CEMENT PONDS STA 1946	U02	54.9	7.5	0.5	6.1	18.9-21.79, 23.16-26.82, 44.81-46.33	#	II
S.R. 83, BRIGHAM CITY - CORRINE	2	35.1	7.5	0.5	1.5	2.74-4.57, 10.67-12.5, 16.76-18.59, 30.48-31.7	100.6	II
FOREST STREET INTERCHANGE	3	36.6	7.5	0.5	1.5	G.W.T.-2.44, 19.05-21.34, 22.86-23.47, 28.96-29.87, 31.7-33.22, 34.75-35.51	2.1	II
SOUTH BRIGHAM CITY INTERCHANGE	U28	36.6	7.5	0.5	1.5	11.89-14.94, 16.46-21.03	#	II
PERRY CANNERY ROAD	3	29.0	7.5	0.5	4.6	5.79-9.75	106.7	II
MASON LANE	2	22.9	7.5	0.5	4.0	7.01-21.79	283.5	II
NORTH MARINA INTERCHANGE	3	18.3	7.5	0.5	6.1	G.W.T.-16.46	121.9	II
NERVA LANE	2	36.6	7.5	0.5	1.2	G.W.T.-11.28, 27.28-35.1	332.2	II
HOT SPRINGS LANE	2	29.0	7.5	0.5	0.0	G.W.T-1.83, 12.19-26.97	58.8	II
HOT SPRINGS INTERCHANGE	5	39.6	7.5	0.5	0.6	13.11-14.33, 17.07-28.04	66.4	II
I-15 IN WEBER COUNTY								
PLAIN CITY INTERCHANGE	3	30.5	7.5	0.5	1.2	G.W.T.-2.13, 23.77-25.6	24.5	II
1700 N.	2	12.2	7.5	0.5	0.6	2.74-3.96, 10.06-11.13	51.8	II
450 N.	2	30.5	7.5	0.5	3.1	4.27-6.71, 19.51-22.25, 23.62-27.13	58.8	II
300 N.	2	25.9	7.5	0.5	0.9	1.22-3.05, 5.49-9.14, 19.51-25.76	136.6	II
700 S.	1	18.3	7.5	0.5	1.5	G.W.T.-5.79, 7.01-7.92	57.9	II
1200 S. INTERCHANGE	1	18.3	7.5	0.5	2.1	G.W.T.-7.32	27.4	II
SOUTHERN PACIFIC RR	2	18.3	7.5	0.5	1.8	G.W.T.-5.79, 6.86-7.92	64.0	II
WEBER RIVER	2	39.6	7.5	0.5	0.9	G.W.T.-2.13, 29.57-35.51	19.2	I
WILSON INTERCHANGE (2100	2	41.2	7.5	0.5	2.4	G.W.T.-5.49, 32.46-33.83	60.4	II

Appendix A - Results of site-specific evaluation of bridge sites (continued)

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
D&RGWRR	1	51.8	7.5	0.5	2.6	3.05-5.49, 9.14-10.36, 21.34-46.63, 50.9-51.82	73.1	II
UPRR STA 1114+68	1	32.0	7.5	0.5	1.5	1.83-3.05, 5.18-14.94, 23.93-29.41	172.2	II
WEST OGDEN INTERCHANGE @ PENNSYLVANIA AVE	2	30.5	7.5	0.5	1.2	1.52-16.46, 24.08-28.35	279.8	II
UPRR STA 1075	1	33.5	7.5	0.5	13.4	14.48-29.87	1.8	II
31ST STREET	2	39.6	7.5	0.5	9.5	20.12-22.56, 37.19-38.56	0.0	II
I-84 CONNECTION	3	27.4	7.5	0.5	7.6	G.W.T.-16.15, 18.9-26.37	95.7	II
4400 S. UNDERPASS	2	21.3	7.5	0.5	8.5	G.W.T.-13.56, 17.98-19.66	77.4	II
RIVERDALE RD.	1	18.3	7.5	0.5	4.0	4.88-17.22	173.7	II
5600 S.	1	18.3	7.5	0.5	5.2	G.W.T.-15.7	94.5	II
UPRR UNDERPASS (HILL	2	29.0	7.5	0.5	8.5	G.W.T.-23.01	101.2	II

I-84 FROM I-80 TO I-15

4400 S.	4	16.8	7.5	0.5	4.0	3.96-7.01, 9.75-14.17	88.4	II
RIVERDALE RD.	1	30.5	7.5	0.5	3.1	G.W.T.-3.96, 7.32-10.06	51.8	II
1050 W.	3	24.4	7.5	0.5	2.1	G.W.T.-3.05, 4.57-5.49, 6.4-7.62	259.1	II
6600 S. OVERPASS	3	21.3	7.5	0.5	1.4	1.83-3.96	33.5	II
SOUTH WEBER	1	21.3	7.5	0.5	9.1	15.85-18.9	0.0	II
WEBER RIVER (NEAR	23	12.2	7.5	0.5	3.7	0-0	0.0	IV
UINTAH JUNCTION INTERCHANGE	18	12.2	7.5	0.5	4.9	0-0	0.0	IV
MOUNTAIN GREEN INTERCHANGE	1-B	18.3	6.5	0.4	8.5	G.W.T.-18.14	152.4	II
COTTONWOOD CREEK	2	15.9	6.5	0.4	0.5	7.01-11.3, 15.2-15.85	53.0	I
PETERSON INTERCHANGE	##							II
STODDARD INTERCHANGE	##							II
MORGAN INTERCHANGE	##							II
WEBER RIVER (DEVILS SLIDE TO HENEFER)	2	12.8	6.5	0.4	3.7	0-0	0.0	IV
DEVIL'S SLIDE INTERCHANGE	3	22.9	6.5	0.4	2.4	4.3-5.33	18.0	II
"O LINE" (DEVILS SLIDE TO HENEFER)	1	15.2	6.5	0.4	4.5	0-0	0.0	IV
HENEFER INTERCHANGE	1	9.1	6.5	0.4	0.9	0-0	0.0	IV
HWY 30 OVERPASS	3	9.1	6.5	0.4	1.2	3.96-5.49	30.0	II

I-15 IN DAVIS COUNTY

CLEARFIELD INTERCHANGE	2	32.0	7.5	0.5	4.9	G.W.T.-5.79, 8.23-9.3, 11.89-15.09, 18.14-21.03, 29.87-30.94	43.9	II
OVERPASS (NO RD. UNDER)	4	25.9	7.5	0.5	10.4	10.67-11.13	0.0	II
HILLFIELD RD.	1	19.8	7.5	0.5	8.2	G.W.T.-18.75	63.1	II
SYRACUSE RD.	1	30.5	7.5	0.5	3.7	7.62-19.2	115.2	II
NORTH LAYTON INTERCHANGE	4	18.3	7.5	0.5	1.2	G.W.T.-2.44, 4.88-6.71, 13.41-17.22	46.3	II
CHURCH ST.	2	19.8	7.5	0.5	2.0	4.11-11.13, 15.54-19.66	90.2	II
GENTILE ST.	2	27.4	7.5	0.5	4.0	6.1-7.01, 7.62-9.91, 11.89-17.98, 19.51-26.37	128.6	II
S. LAYTON INTERCHANGE	3	19.8	7.5	0.5	1.1	1.22-8.08, 10.52-11.28, 16.46-18.75	128.6	II
KAYSVILLE INTERCHANGE	2	30.5	7.5	0.5	4.6	G.W.T.-15.24	304.8	II
BURTON LANE	1	19.8	7.5	0.5	1.2	G.W.T.-3.05, 6.1-8.53, 13.26-18.75	61.0	II
SHEPARDS LANE	1	29.0	7.5	0.5	3.7	3.81-7.16, 10.36-11.89, 13.41-25.91	86.9	II

Appendix A - Results of site-specific evaluation of bridge sites (continued)

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
SB OFF RAMP TO HWY 89 (L LINE)	1	21.3	7.5	0.5	0.9	G.W.T.-13.11, 16.76-19.2	228.9	II
HWY 89 OVER I-15	1	27.4	7.5	0.5	0.5	7.92-10.67, 17.98-19.51, 21.03-23.77	108.5	II
STATE ST. FARMINGTON	3	24.4	7.5	0.5	2.4	3.66-6.86, 9.14-9.75, 17.68-21.79	24.4	II
WALKER LANE	1	27.4	7.5	0.5	1.5	G.W.T.-5.79, 14.33-16.15, 19.51-24.84	75.7	II
GLOVER LANE	2	30.5	7.5	0.5	0.8	14.63-25.0	13.1	II
PARRISH LANE	1	38.1	7.5	0.5	1.5	12.5-15.24, 16.46-17.07, 18.29-25.3	42.4	II
PAGES LANE BOUNTIFUL	U7	18.3	7.5	0.5	0.0	1.52-2.74, 7.62-13.11, 15.54-16.46	#	II
1000 N. BOUNTIFUL	U4	19.2	7.5	0.5	0.0	G.W.T.-3.35, 8.23-17.22	#	II
400 N. BOUNTIFUL	U19	27.4	7.5	0.5	3.1	4.27-7.62, 8.84-17.37, 18.9-23.01	#	II
500 S. BOUNTIFUL	U5	22.9	7.5	0.5	12.2	G.W.T.-14.63, 15.85-17.68, 18.9-20.73	#	II
1500 S. BOUNTIFUL	U4	21.3	7.5	0.5	12.2	G.W.T.-13.41, 14.63-16.15, 17.37-20.27	#	II
1100 W. N. SALT LAKE (HOWARD RD?)	U3	25.9	7.5	0.5	2.7	G.W.T.-15.85, 21.34-22.56, 23.47-24.69	#	II
HWY 89 OVER I-15						##		III
I-80 FROM I-15 TO WYOMING BORDER								
200 W.	1B1	39.6	7.5	0.5	0.0	3.66-4.88, 13.11-14.33, 18.9-22.86, 25.6-27.43, 31.7-32.92	35.7	II
UPRR NEAR 150 W.	2B2	33.5	7.5	0.5	0.0	1.83-3.66, 12.19-13.72, 23.16-28.96, 29.87-33.53	43.9	II
W. TEMPLE	3B2	35.1	7.5	0.5	1.2	19.2-21.03, 31.39-35.05	0.0	II
MAIN STREET	4B2	32.0	7.5	0.5	1.2	G.W.T.-2.13, 12.19-19.51, 25.6-32	105.7	II
STATE STREET	5B2	30.5	7.5	0.5	2.4	24.08-30.48	0.0	II
300 E.	6B2	35.1	7.5	0.5	4.0	G.W.T.-10.21, 22.25-30.18, 34.44-34.9	123.8	II
500 E.	7B2	24.4	7.5	0.5	1.1	1.22-4.27, 14.94-16.46, 19.51-20.42, 22.56-23.47	118.9	II
700 E.	8B1	24.4	7.5	0.5	3.1	G.W.T.-10.36, 11.28-13.11, 17.07-20.57, 22.56-24.38	287.1	II
900 E.	9B2	21.3	7.5	0.5	4.3	G.W.T.-11.58, 16.46-17.68	183.8	II
KIMBALL JUNCTION	2	12.2	6.5	0.4	9.1	0-0	0.0	IV
"D LINE" (BETWEEN KIMBALL JCT. AND SILVER CR. JCT.)	2	9.1	6.5	0.4	0.0	0-0	0.0	IV
SILVER CREEK JCT. INTERCHANGE	1	9.1	6.5	0.4	2.3	0-0	0.0	IV
I-80 FROM I-15 TO NEVADA BORDER								
1000 WEST						##		III
JORDAN RIVER	A-3	30.5	7.5	0.5	0.0	G.W.T.-3.05, 10.06-12.8, 17.07-18.9	118.9	I
NAVAJO ST.						##		III
200 SOUTH						##		III
UPRR						##		III
REDWOOD RD.	A-7	36.6	7.5	0.5	0.0	G.W.T.-6.1, 9.75-11.28, 14.63-16.76, 21.34-24.38, 29.87-33.99	204.2	II
NORTH PT. CONSOLIDATED CANAL	A-9	30.5	7.5	0.5	0.0	8.84-10.67, 21.03-22.86, 29.57-30.48	116.1	II

Appendix A - Results of site-specific evaluation of bridge sites (continued)

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
3400 W.	A-14	30.5	7.5	0.5	0.0	10.67-17.07, 18.9-19.81, 24.38-26.82, 28.65-30.48	76.2	II
40TH W RAMP Q-1	3	30.8	7.5	0.5	1.5	6.4-9.45, 12.19-19.81	107.3	II
5600 W.	1	32.0	7.5	0.5	1.5	3.05-3.96, 7.47-8.53, 24.08-27.43	104.2	II
RR (APPROX. 6200 W.)	1	30.5	7.5	0.5	0.9	9.45-10.36, 13.72-15.24, 17.07-21.95, 25.91-27.43, 28.35-29.72	50.8	II
7200 W.	2	33.5	7.5	0.5	0.3	1.22-2.74, 4.27-8.53, 10.06-33.53	196.3	II
10400 W.	2	30.5	7.5	0.5	0.3	14.94-29.41	0.0	II
SALTAIR INTERCHANGE	##							III
BLACK ROCK INTERCHANGE	1	33.5	7.0	0.5	5.8	7.01-7.92, 9.14-14.17, 17.37-19.81, 21.34-21.95, 25.91-26.37	41.0	II
LAKEPOINT INTERCHANGE @ 5217+50	1	27.4	6.5	0.4	1.2	8.53-17.07, 19.2-22.25	426.7	II
LESLIE SALT PLANT @ 5205+29	2	42.7	6.5	0.4	1.5	12.19-41.61	84.1	II
NEAR LESLIE SALT PLANT @ 5131+13	1	30.5	6.5	0.4	0.0	G.W.T.-0.91, 28.04-29.87	36.6	II
WARNER SPUR @ 4676+76	3	22.9	6.5	0.4	0.5	3.05-10.06, 12.5-19.20	277.0	II
BURMESTER COUNTY RD. @ 4626+38	1	27.4	6.5	0.4	1.2	G.W.T.-1.52, 4.88-14.33, 16.46-24.99	313.9	II
SOLAR SALT PLANT @	4	21.3	6.5	0.4	1.1	1.52-4.27	70.0	II
DUGWAY RD. NEAR TIMPIE	6	36.6	6.5	0.4	1.5	G.W.T.-4.88, 15.85-17.07, 28.8-32	204.2	II
BRIDGES EAST OF DELLE #2	2	30.5	6.5	0.35	0.9	2.74-10.06, 19.81-27.89	249.6	II
BRIDGES EAST OF DELLE #1	1	15.2	6.5	0.35	1.1	G.W.T.-3.35, 4.27-5.49	137.2	II
DELLE INTERCHANGE	3	35.1	6.5	0.35	8.5	9.1-19.05, 29.87-35.05	107.0	II
KNOLLS INTERCHANGE	3	24.4	6.5	0.3	9.1	G.W.T.-10.97	39.0	II
WENDOVER INTERCHANGE	2	36.6	6.5	0.3	3.1	G.W.T.-4.27, 8.84-9.45, 14.33-16.76, 19.81-22.25	76.0	II

I-15 CORRIDOR IN SALT LAKE COUNTY

I-15 OVER BECK ST. AND RR	##							III
800 WEST EXIT	##							III
NORTH TEMPLE ST.	DH-10	26.2	7.5	0.5	2.4	2.74-8.84, 20.12-26.21	117.4	II
STRUCTURES 48-58 (300 S. TO 100 S.)	P42	31.1	7.5	0.5	1.8	G.W.T.-3.05, 4.57-6.1, 11.89-13.72, 17.98-25.91, 29.57-34.14	#	II
	P87	31.1	7.5	0.5	1.8	G.W.T.-3.05, 4.27-5.49, 7.32-12.19, 16.46-21.34, 22.56-25.3, 27.43-31.09	#	II
	P199	31.1	7.5	0.5	1.8	G.W.T.-3.05, 3.35-4.57, 18.29-21.34, 26.52-29.57	#	II
	P203	31.1	7.5	0.5	1.8	17.68-23.77	#	II
STRUCTURES 44-47 (AT 400 S.)	P81	31.1	7.5	0.5	1.8	G.W.T.-3.05, 3.66-6.1, 7.92-10.06, 13.72-18.29, 19.81-21.34, 22.56-30.48	#	II
	P186	31.1	7.5	0.5	1.8	G.W.T.-4.88, 15.24-16.76, 19.81-25.6	#	II
STRUCTURES 71 & 72 (500 S. & 600 S. VIADUCTS)	P230	31.1	7.5	0.5	1.8	G.W.T.-3.05, 18.29-21.34	#	II
	P242	31.1	7.5	0.5	1.8	3.05-3.66, 9.75-12.19, 15.24-18.9, 19.81-21.34, 24.99-28.35	#	II
STRUCTURES 39-43 (700 S. TO 500 S.)	P180	31.1	7.5	0.5	1.8	G.W.T.-3.66, 7.62-9.14, 13.41-20.73, 22.25-30.18	#	II

Appendix A - Results of site-specific evaluation of bridge sites (continued)

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
	P229	31.1	7.5	0.5	1.8	G.W.T.-3.05, 6.1-6.71, 14.63-17.68, 22.86-24.38, 26.82-30.48	#	II
	P240	31.1	7.5	0.5	1.8	6.1-7.01, 18.29-19.2, 28.96-31.09	#	II
	DH-14	18.9	7.5	0.5	1.8	3.05-7.01, 13.41-16.46	143.3	II
	DH-15 / DH-15A	31.1	7.5	0.5	0.6	1.22-3.96, 17.06-17.67, 24.38-25.6, 26.5-27.74, 28.65-31.09	33.5	II
STRUCTURES 28,29,68, 69,70 (1300 S. TO 900 S. OFF-RAMP)	P244	31.1	7.5	0.5	1.8	G.W.T.-3.66, 21.34-28.04	#	II
	P246	31.1	7.5	0.5	1.8	G.W.T.-3.05, 3.96-5.49, 18.9-20.73, 25.6-26.82	#	II
	P249	26.5	7.5	0.5	1.8	6.71-12.19, 18.9-20.73	#	II
	P250	26.5	7.5	0.5	1.8	3.05-4.27	#	II
STRUCTURES 36 & 37 (AT 800 S.)	P10	37.2	7.5	0.5	1.8	G.W.T.-5.18, 7.92-11.58, 15.85-25.6, 27.74-29.57, 31.09-33.22, 34.75-37.19	#	II
	P175	31.1	7.5	0.5	1.8	3.05-12.19, 16.76-19.81, 24.38-27.43, 28.35-30.48	#	II
STRUCTURES 33 & 34 (AT 900 S. & RR)	P170	31.1	7.5	0.5	1.5	G.W.T.-3.05, 7.62-8.53, 10.67-11.58, 17.07-22.86	#	II
	DH-13	38.4	7.5	0.5	1.5	8.84-10.36, 16.76-21.95, 23.77-26.82, 35.05-36.58	19.9	II
STRUCTURES 30 & 31 (AT RR & BROOKLYN AVE.)	P166	31.1	7.5	0.5	1.5	21.34-27.43	#	II
	DH-16	34.1	7.5	0.5	2.4	G.W.T.-3.96, 7.62-8.53, 13.41-17.07, 25.3-34.14	88.4	II
STRUCTURES 27A & 27B (AT 400 W.)	P160	31.1	7.5	0.5	1.5	2.44-6.71, 20.12-23.47, 30.48-31.09	#	II
	P161	31.1	7.5	0.5	1.5	3.05-4.27, 9.75-13.41, 21.34-25.3, 27.74-28.35	#	II
STRUCTURES 23-26 (AT 1300 S.)	P61	24.1	7.5	0.5	1.5	10.67-13.72, 15.24-16.76, 24.69-25.91, 27.43-28.96	#	II
	P155	31.1	7.5	0.5	1.5	2.44-4.27, 9.14-14.33, 22.56-26.52, 29.87-31.09	#	II
STRUCTURES 15-18 (AT 1700 S.)	P53	31.1	7.5	0.5	1.5	G.W.T.-3.66, 8.23-14.02, 21.34-25.3	#	II
	P146	31.1	7.5	0.5	1.5	G.W.T.-3.96, 7.62-15.85, 21.34-24.08	#	II
STRUCTURES 11-14 (AT 2100 S.)	P3	29.9	7.5	0.5	1.5	G.W.T.-3.05, 10.97-16.15, 20.73-29.87	#	II
	P51	29.9	7.5	0.5	1.5	15.24-18.29, 27.43-29.87	#	II
STRUCTURES 7-10 (OVER RR SO. OF 2100 S.)	P2	27.4	7.5	0.5	1.5	G.W.T.-2.44, 12.19-13.11, 18.29-22.56, 24.38-25.3	#	II
	P50	31.1	7.5	0.5	1.5	12.8-18.29, 21.34-28.65	#	II
	DH-4	27.4	7.5	0.5	0.0	G.W.T.-4.27, 10.97-11.58, 19.2-21.64, 22.86-25.3	205.6	II
	DH-9	25.0	7.5	0.5	0.0	9.14-9.75, 10.67-11.58, 14.32-14.94, 15.85-16.46, 21.03-22.86	#	II
STRUCTURES 1,2,3 (N OF 2700 S. & SO. OF RR) (I-80 INTERCHANGE)	P47	28.0	7.5	0.5	0.9	G.W.T.-3.05, 4.57-5.18, 18.29-22.86	#	II
	P114	26.5	7.5	0.5	0.6	G.W.T.-3.66	#	II

Appendix A - Results of site-specific evaluation of bridge sites (continued)

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
	DH-5	27.4	7.5	0.5	5.5	G.W.T.-6.1, 23.16-27.43	4.3	II
	DH-7	22.0	7.5	0.5	0.6	16.15-17.07, 18.59-21.95	36.6	II
	DH-8	25.0	7.5	0.5	1.5	14.94-16.15, 19.51-23.47	10.4	II
2700 S.	U2	25.3	7.5	0.5	1.2	2.74-4.27, 13.41-17.98, 19.81-25.3	#	II
3300 S.	DH-11	29.0	7.5	0.5	1.8	G.W.T.-2.44, 10.97-13.41, 15.54-18.59, 21.34-23.16, 26.52-28.96	21.3	II
3900 S. UNDERPASS	U4	29.0	7.5	0.5	3.1	G.W.T.-15.24, 15.54-16.46, 17.98-22.56	#	II
D&RGW RAILROAD (200 W. / 400 W.)	U3	45.7	7.5	0.5	0.9	6.4-7.01, 9.14-9.75, 11.58-15.24, 20.12-31.7, 39.32-45.72	#	II
4500 S.	U1	25.9	7.5	0.5	1.5	1.83-3.05, 3.96-7.32, 7.92-9.14, 9.75-11.58, 19.2-20.12, 21.64-24.08, 24.99-25.91	#	II
4800 S.	U1	17.7	7.5	0.5	2.1	G.W.T.-6.71, 7.32-11.28	#	II
GERMANIA ST .	U3	25.3	7.5	0.5	3.1	4.27-5.18, 7.32-8.53, 11.89-13.41, 17.98-19.51	#	II
5300 S.	U4	23.8	7.5	0.5	4.0	5.79-7.32, 15.54-20.12	#	II
5900 S.	U1	16.5	7.5	0.5	0.9	G.W.T.-3.05	#	II
	DH-12	15.2	7.5	0.5	7.6	G.W.T.-8.53	0.0	II
6400 S.	H-1	22.9	7.5	0.5	3.7	7.32-10.36, 21.64-22.86	#	II
	H-5	18.3	7.5	0.5	2.4	4.57-8.53	#	II
7200 S.	5 (G)	16.8	7.5	0.5	6.1	G.W.T.-8.23	#	II
CENTER ST. MIDVALE (7720	13	22.9	7.5	0.5	6.1	G.W.T.-21.34	#	II
WASATCH ST. (8000 S.)	17	21.6	7.5	0.5	3.1	6.1-9.14, 13.41-15.24, 16.76-20.42	#	II
9000 S.	22	22.3	7.5	0.5	0.6	5.18-7.01	#	II
10000 S.	DH-1	18.3	7.5	0.5	1.8	G.W.T.-4.57, 12.19-17.07	228.6	II
10600 S.	36	24.1	7.5	0.5	1.5	22.56-23.47	#	II
11400 S.	DH-17	15.2	7.5	0.5	5.8	G.W.T.-12.19	88.7	II
12300 S.	3	27.4	7.5	0.5	1.5	2.74-7.62, 9.14-10.67, 11.28-12.19, 16.76-19.66,	112.8	II

I-215 (BELT ROUTE)

NORTHBOUND LANE OVER I-15	##							III
RR NEAR NORTH I-15 INTERCHANGE	1A	30.8	7.5	0.5	5.5	G.W.T.-8, 9.75-25.5	649.2	II
REDWOOD RD. (NORTH)	3	44.8	7.5	0.5	2.1	7-8.5, 10.4-13.3, 26.8-33.5	91.7	II
JORDAN RIVER (NORTH)	S-17	31.1	7.5	0.5	0.9	5.5-7.2, 7.9-10.6, 21.65-23.4	481.6	I
2200 N.	5	32.6	7.5	0.5	1.2	8.8-9.7, 15.7-16.8, 18.75-23.1, 25-25.6	9.1	II
1700 N.	4	34.1	7.5	0.5	1.4	G.W.T.-13.5	222.5	II
600 N.	1	43.3	7.5	0.5	0.3	G.W.T-9.4, 20.42-29.9, 41.8-43.1	213.4	II
NORTH TEMPLE	4	30.5	7.5	0.5	1.2	4.9-6.6, 15.6-18.2, 25.4-27	53.1	II
I-80, C, Y, & M RAMPS	3	30.5	7.5	0.5	1.5	3-6, 9.5-12.1, 13.4-18, 24.4-27.9	158.5	II
RR (APPROX. 400 S.)	6	32.0	7.5	0.5	0.3	11.6-13.5, 19.2-23, 29.7-31.6	9.1	II
500 S.	2	32.0	7.5	0.5	1.2	1.4-1.7, 2.7-15.2, 16.8-18.9, 25.6-28	360.9	II
INDIANA AVE	3	32.0	7.5	0.5	0.6	10.97-12.19, 13.4-15.5, 18.29-29.87	46.0	II
SURPLUS CANAL	4	30.5	7.5	0.5	1.2	5.2-11.5, 22.6-23.4, 25.2-29.4	132.0	II
CALIFORNIA AVE	3	27.4	7.5	0.5	0.9	G.W.T.-2.13, 7.16-16.15, 21.34-23.47	110.0	II
1700 S.	3	22.9	7.5	0.5	2.1	15.8-19.5	0.0	II
21OO S. INTERCHANGE	1 (1962	26.5	7.5	0.5	1.5	G.W.T.-2.6, 6.7-10, 11.3-14.6	152.4	II

Appendix A - Results of site-specific evaluation of bridge sites (continued)

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
2700 S.	3	28.0	7.5	0.5	1.7	9.5-12.65, 14.33-16.31, 17.68-19.5, 23.8-24.99	106.7	II
3100 S.	1	22.0	7.5	0.5	0.0	2.4-3.6, 6.1-7.6, 8.8-13.1, 14.6-17	452.9	II
3500 S.	2	28.0	7.5	0.5	0.8	3.96-21.64	137.2	II
3800 S.	2	24.4	7.5	0.5	1.5	4.42-5.18, 7.32-9.14, 21.64-22.56	37.2	II
4100 S.	1	26.5	7.5	0.5	1.2	7.62-12.19, 18.75-21.64, 23.47-24.38	71.0	II
4700 S.	1	30.5	7.5	0.5	8.2	G.W.T-10.67, 23.47-29.41	146.3	II
5400 S.	1	18.3	7.5	0.5	6.4	7.92-10.52, 16.61-17.53	46.3	II
REDWOOD RD. (SOUTH)	3	25.9	7.5	0.5	2.1	5.18-7.92, 8.23-10.97, 18.29-20.57	30.5	II
1300 W.	2	24.4	7.5	0.5	1.2	4.88-5.64, 8.84-10.06, 12.8-14.32	172.2	II
RIVERSIDE DR.	3	16.8	7.5	0.5	2.1	2.59-4.27	36.6	II
JORDAN RIVER (SOUTH)	3	15.2	7.5	0.5	0.9	1.98-3.05	6.1	I
700 W.						##		III
6400 S.	1	18.3	7.5	0.5	4.0	G.W.T.-7.92, 8.99-10.97, 13.41-14.02	79.2	II
STATE STREET	3	21.3	7.5	0.5	5.5	7.01-10.36, 15.54-16.15	49.7	II
700 E.	1	22.9	7.5	0.5	13.1	21.03-22.86	0.0	II
900 E.	3	27.4	7.5	0.5	13.7	13.72-17.98	8.5	II
1300 E.	3	21.3	7.5	0.5	3.7	6.1-12.19	213.4	II
2300 E.	3	22.9	7.5	0.5	0.0	9.45-15.24	9.1	II

I-15 IN UTAH COUNTY

HWY 89 & RR NORTH OF MAIN LEHI	5	18.3	7.5	0.5	1.2	G.W.T.-1.52	9.1	II
MAIN ST. LEHI	1J2	19.8	7.5	0.5	0.6	0-0	#	IV
MAIN ST. AMERICAN FORK	1K3	24.4	7.5	0.5	0.0	G.W.T.-1.83	#	II
300 W. / 200 S. AMERICAN FORK	1L2	18.3	7.5	0.5	3.4	5.49-11.28, 12.8-15.7	#	II
100 E. AMERICAN FORK	1N1	24.4	7.5	0.5	0.3	G.W.T.-2.13, 11.3-12.8	#	II
500 E. AMERICAN FORK	60	18.3	7.5	0.5	0.0	11.58-14.63	#	II
6800 N. (COUNTY RD.)	2B3	30.5	7.5	0.5	0.0	9.3-11.28, 27.89-28.19	#	II
COUNTY RD.	2C4	24.4	7.5	0.5	0.6	10.36-13.72	#	II
2000 W. LINDON	2D3	18.3	7.5	0.5	0.3	0-0	#	IV
GENEVA RD. (HWY 114) AND UPRR	2E3	42.7	7.5	0.5	0.0	1.22-3.5, 40.54-42.67	#	II
1600 N. OREM	2F3	30.5	7.5	0.5	0.0	G.W.T.-3.05, 26.21-29.87	#	II
CENTER ST. OREM	118A	21.3	7.5	0.5	0.0	G.W.T.-2.74, 5.33-8.84, 10.36-10.67, 14.32-21.34	#	II
400 S. OREM	3E3	24.4	7.5	0.5	0.9	1.07-2.13, 11.58-13.11, 23.77-24.38	#	II
1200 S. OREM	132A	21.3	7.5	0.5	0.6	10.67-13.26	#	II
2000 S. OREM	3I2-2	30.5	7.5	0.5	0.0	7.01-8.84	#	II
BRIDGES OVER UPRR AND RG&WRR	157a	30.5	7.5	0.5	0.0	G.W.T.-4.88, 7.62-13.41, 20.12-23.47, 25.76-30.48	#	II
	5	21.3	7.5	0.5	0.3	G.W.T.-6.71, 8.84-13.41	197.0	II
BRIDGE OVER UPRR	3K3	30.5	7.5	0.5	0.3	G.W.T.-5.49, 9.14-10.06, 12.19-13.72	#	II
	3	18.9	7.5	0.5	0.3	G.W.T.-7.01, 9.14-13.41	239.3	II
900 N. PROVO	3L4	24.4	7.5	0.5	0.6	3.96-6.71	#	II
	1	15.2	7.5	0.5	0.6	G.W.T.-6.71, 10.06-13.41	189.0	II
PROVO RIVER	3M2	30.5	7.5	0.5	1.5	0-0	#	IV
CENTER STREET PROVO	3Q4	30.5	7.5	0.5	1.5	8.23-14.38	#	II

Appendix A - Results of site-specific evaluation of bridge sites (continued)

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
600 S. PROVO	3S7	9.1	7.5	0.5	0.9	G.W.T.-1.42	#	II
APROX. 950 S.						##		III
UNIVERSITY INTERCHANGE	33A	30.5	7.5	0.5	0.0	2.44-4.88, 6.71-7.32, 16.46-28.04	61.0	II
IRONTON INTERCHANGE	24A	33.5	7.5	0.5	1.2	G.W.T.-1.52, 4.27-5.18, 22.86-24.99	55.2	II
SPRINGVILLE 400 S.	14A	33.5	7.5	0.5	2.4	4.88-9.75, 21.34-23.47, 25.3-32.16	194.8	II
LEMAR LANE	5A	36.6	7.5	0.5	0.6	0.91-3.05, 4.88-6.4, 14.63-16.15, 25.6-31.7, 34.9-35.81	61.0	II
UPRR NORTH OF SPANISH FORK	U6	36.6	7.5	0.5	2.4	G.W.T.-4.27, 32.61-33.83, 34.75-36.42	34.1	II
RR N. OF SPANISH FORK	U11	39.6	7.5	0.5	1.5	G.W.T.-5.33, 8.84-10.36, 14.94-22.86, 25.6-29.26, 31.7-33.53, 37.49-39.17	73.2	II
MOARK INTERCHANGE	3	45.7	7.5	0.5	2.1	G.W.T.-8.53, 23.77-34.75, 35.36-45.57	122.8	II
MAIN ST. SPANISH FORK	U21	29.0	7.5	0.5	0.9	G.W.T.-1.22, 2.74-8.84, 23.77-27.89	#	II
300 W. OVERPASS, SPANISH FORK	U29	25.9	7.5	0.5	3.1	6.1-14.63	#	II
400 N. SPANISH FORK	U43	24.4	7.5	0.5	1.5	4.27-10.36	#	II
SPANISH FORK SPUR (RR)	U57	15.2	7.5	0.5	1.5	5.49-10.06, 11.58-12.65	#	II
LELAND-BENJAMIN RD.	U61	24.4	7.5	0.5	2.1	2.44-7.62, 8.84-12.19, 13.41-19.2, 19.81-22.86	#	II
SPANISH FORK RIVER	U68	24.4	7.5	0.5	2.1	3.66-12.19, 12.8-16.76, 17.07-24.08	#	I
ARROWHEAD RESORT RD. BENJAMIN	U85	30.5	7.5	0.5	2.1	G.W.T.-3.35, 8.23-10.06, 11.89-12.5, 15.85-18.29	#	II
LELAND BENJAMIN RD. STA 666+30	U114	24.4	7.5	0.5	3.1	G.W.T.-6.71, 7.32-23.32	#	II
1000 N. PAYSON	14-1	24.4	7.5	0.5	0.9	G.W.T.-4.88, 16.61-17.37, 19.2-20.12	152.4	II
400 N. PAYSON	15-1	24.4	7.5	0.5	4.0	G.W.T.-7.62, 15.24-16.76	57.8	II
UTAH AVENUE PAYSON	16-6	24.4	7.5	0.5	4.3	G.W.T.-6.71, 9.14-11.13	3.0	II
800 S. PAYSON	17-5	27.4	7.5	0.5	5.2	G.W.T.-7.01, 10.36-14.33, 18.29-18.9, 21.34-23.16	113.9	II
1100 S. PAYSON	17-1	27.4	7.5	0.5	3.4	6.4-11.89, 17.07-19.81	63.4	II
STRUCTURE 18 STA. 311+16	18-2	24.4	7.5	0.5	1.5	G.W.T.-3.35, 7.01-9.14, 13.72-21.03	135.6	II
BRIDGE AT STA 284+52	4	24.4	7.5	0.5	1.5	1.83-6.1, 9.14-10.97, 11.58-17.37	193.5	II

I-15 AREAS SOUTH OF UTAH COUNTY

FRONTAGE RD. "A" (WEST LEVAN TO S. NEPHI)	1	30.5	7.1	0.43	7.0	0-0	0.0	IV
WEST LEVAN INTERCHANGE	1	30.5	7.1	0.43	5.2	8.38-11.89, 16.15-17.53	34.7	II
UPRR (SEVIER RIVER TO MILLS JCT.)	3	24.4	7.1	0.43	2.1	10.67-11.28, 13.72-14.63, 15.24-16.15, 17.37-20.73	118.9	II
YUBA STATE PARK INTERCHANGE	3	24.4	6.5	0.40	6.1	10.67-12.8	40.5	II
SEVIER RIVER BRIDGE	3	19.8	6.5	0.40	2.4	G.W.T.-3.66	13.0	I
"X" LINE (STA716+30) (SCIPIO TO SEVIER RIVER)	1	30.5	6.5	0.40	1.5	0-0	0.0	IV
"I LINE" (STA711+80) (FILLMORE TO JUAB CO.	4	15.2	6.5	0.40	6.7	0-0	0.0	IV

Appendix A - Results of site-specific evaluation of bridge sites (continued)

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
"L-LINE" (BAKER CANYON TO MEADOW)	1	34.8	6.5	0.40	4.9	6.1-9.45, 14.33-18.29	100.3	II
"A LINE"(STA 400+00) BEAVER	3	21.3	6.5	0.40	3.1	0-0	0.0	IV
BEAVER RIVER	1	9.1	6.5	0.40	1.5	0-0	0.0	IV
"L LINE"(STA 462+72) BEAVER	2	7.6	6.5	0.40	1.8	0-0	0.0	IV
"J LINE"(STA 490+52) BEAVER	2	7.6	6.5	0.40	3.1	0-0	0.0	IV
"K LINE"(STA 518+60) BEAVER	1	7.6	6.5	0.40	3.1	0-0	0.0	IV
"G LINE" (SUMMIT TO PARAGONAH)	DH1	15.2	6.5	0.40	8.8	0-0	0.0	IV
PARAGONAH INTERCHANGE	DH1	35.4	6.5	0.40	5.5	0-0	0.0	IV
"G LINE" (PARAGONAH TO FREMONT WASH)	2	30.5	6.5	0.40	5.2	5.79-8.84, 10.36-12.5, 14.94-21.03, 21.95-24.08, 24.99-27.13	108.5	II
SOUTH PAROWAN INTERCHANGE	1	23.5	6.5	0.40	10.7	11.28-15.85	82.3	II
"B LINE" (STA221+00) (WASHINGTON-IRON CO. LINE TO HAMILTON FORT)	3	18.3	6.5	0.40	11.9	G.W.T.-13.26	30.5	II
I-70								
HWY 89	8	25.9	6.5	0.40	9.0	10.36-15.85	137.2	II
SEVIER RIVER	6	16.8	6.5	0.40	1.5	1.52-1.98	12.2	I
SOUTH SALT WASH	1	15.2	6.5	0.30	9.5	0-0	0.0	IV
GREEN RIVER	2	7.6	6.1	0.10	3.1	0-0	0.0	IV
I-15 IN ST. GEORGE AREA								
WASHINGTON INTERCHANGE	4	12.2	6.5	0.40	3.4	G.W.T.-9.15	121.9	II
"C LINE" ST. GEORGE TO MIDDLETON	3	10.7	6.5	0.40	0.8	0-0	0.0	IV
100 S. IN ST. GEORGE	3	9.1	6.5	0.40	4.1	0-0	0.0	IV
700 S. IN ST. GEORGE	2	10.7	6.5	0.40	4.6	0-0	0.0	IV
SANTA CLARA RIVER BRIDGE	U5	10.7	6.5	0.40	3.1	G.W.T.-4.57, 7.47-10.67	#	I
VIRGIN RIVER BRIDGE	U3	21.3	6.5	0.40	0.6	G.W.T.-8.84, 9.75-16.15	#	I
FEDERAL HIGHWAYS								
HWY 6 OVER SPANISH FORK RIVER	##							I
ALL BRIDGES ON HWY 40 FROM WASATCH FRONT TO NEVADA	##							III
HWY 89/91 OVER LOGAN RIVER	##							I
HWY 89/ 91 OVER LITTLE BEAR RIVER	DH3	30.5	7.1	0.45	0.0	1.52-3.05, 9.45-11.28, 21.34-23.47	72.3	I
HWY 89/91 OVER OGDEN RIVER	1	39.6	7.5	0.5	5.2	5.49-6.71, 8.99-10.06, 16-21.03, 31.7-33.53	29.6	I
HWY 89/91 OVER PROVO RIVER	1	25.9	7.5	0.5	3.4	7.16-16.76	100.6	I
HWY 89 OVER SPANISH FORK RIVER	2	11.3	7.1	0.47	1.1	1.22-5.18, 9.14-11.28	204.2	I

Appendix A - Results of site-specific evaluation of bridge sites (continued)

BRIDGE LOCATION	BORING NUMBER	HOLE DEPTH (m)	MAGNITUDE	ACCELERATION	DEPTH TO G.W.T. (m)	LIQUEFIABLE LAYERS (m)	SETTLEMENT IN UPPER 15m (mm)	FUTURE STUDY PRIORITY
HWY 89 OVER SAN PITCH RIVER	2	21.3	6.5	0.4	3.4	0-0	0.0	IV
HWY 89 OVER CLEAR CREEK	1	9.1	6.5	0.4	1.5	G.W.T.-4.42, 7.32-8.53	91.4	I
HWY 89 OVER SEVIER RIVER 0.5 MILES SOUTH OF SEVIER JCT.	2	12.2	6.5	0.4	1.5	0-0	0.0	IV
HWY 89 OVER SEVIER RIVER APPROX. 2 MILES SOUTH OF SEVIER JCT.	1	9.1	6.5	0.4	0.3	0-0	0.0	IV
HWY 89 OVER SEVIER RIVER NEAR SR-62 JCT	2	6.1	6.5	0.4	1.5	G.W.T.-2.53	42.7	I
HWY 89 OVER HOG CREEK	1	15.2	6.5	0.4	7.0	7.62-12.59, 13.56-14.94	182.9	I
STATE ROADS 30 AND 201 (2100 S. FREEWAY)								
SR-30 OVER MALAD RIVER	1	38.1	7.5	0.5	3.4	32-34.75, 36.27-37.03	0.0	I
SR-30 OVER LITTLE BEAR RIVER	2	38.1	7.1	0.45	1.5	2.13-5.49, 31.7-33.22	160.7	I
2100 S OVER RR	2	22.9	7.5	0.5	1.2	2.74-9.14, 13.72-15.24	161.5	II
800 W. AND 2100 S (HWY 201)	1	28.0	7.5	0.5	0.0	G.W.T.-0.91, 4.27-7.32, 10.36-16.76, 18.9-20.12, 23.77-28.04	262.1	II
2100 S (HWY 201) OVER JORDAN R	1	25.9	7.5	0.5	0.9	G.W.T.-6.1, 9.45-11.89, 12.95-16.76, 20.73-24.08	216.4	I
REDWOOD RD. AND HWY 201 (2100S)	1	24.4	7.5	0.5	0.6	G.W.T.-2.44, 6.4-9.45, 11.28-14.63, 19.2-19.66	280.4	II
2700 W AND 2100 S	##							III
3200 W AND 2100 S (HWY 201)	3	33.5	7.5	0.5	0.9	3.05-3.35, 10.67-11.13, 15.24-18.29, 25.6-26.82, 30.48-30.94	79.2	II
3600 W AND 2100 S	##							III
4000 W AND 2100 S (HWY 201)	2	30.5	7.5	0.5	1.5	1.83-4.88, 8.84-10.06, 11.89-15.24	140.2	II
5600 W AND 2100 S (HWY 201)	1	30.5	7.5	0.5	1.5	G.W.T.-1.83, 4.57-10.97, 28.65-29.41	246.9	II
7200 W AND 2100 S	##							III
8000 W AND 2100 S	##							III
8400 W AND 2100 S	##							III

G.W.T. - GROUNDWATER TABLE

- CONFIDENCE IN RESULTS IS LOW DUE TO NON-STANDARD SAMPLING TECHNIQUES

- BOREHOLE DATA NOT AVAILABLE; HIGH HAZARD DUE TO REGIONAL SCREENING CRITERIA

I - CONFIRMED PRESENCE OF LIQUEFIABLE SEDIMENTS WITH BRIDGE LOCATED AT RIVER CROSSING
-- VERY HIGH PRIORITY FOR FURTHER INVESTIGATION

II - CONFIRMED PRESENCE OF LIQUEFIABLE SEDIMENTS WITH BRIDGE NOT LOCATED AT RIVER CROSSING
OR
INSUFFICIENT INFORMATION TO ASSESS LIQUEFACTION SUSCEPTIBILITY WITH BRIDGE LOCATED AT RIVER CROSSING
-- HIGH PRIORITY FOR FURTHER INVESTIGATION

III - INSUFFICIENT INFORMATION TO ASSESS LIQUEFACTION SUSCEPTIBILITY
-- MODERATE PRIORITY FOR FURTHER INVESTIGATION

IV - LOW LIQUEFACTION HAZARD -- LOW PRIORITY FOR FURTHER ANALYSIS OR MITIGATION

APPENDIX B - DAMES & MOORE GUIDELINES FOR LIQUEFACTION HAZARD EVALUATION OF I-15 CORRIDOR

7.0 Guidelines for Liquefaction Hazard Evaluation

Liquefaction potential maps developed by Anderson et al. (1985) for Salt Lake County indicate that the entire I-15 corridor from 13800 South to 600 North is classified as having a moderate to high liquefaction potential. Thus, according to these maps there is a 10 to 50 percent probability that the critical ground accelerations required to induce liquefaction will be exceeded in 100 years for the entire corridor. A high liquefaction potential has been assigned from about 5000 South northward. Therefore, it is prudent to evaluate liquefaction on a site-specific basis along the corridor. Youd (1996) has identified liquefaction prone areas along the I-15 corridor and that study should be used to assess whether additional analyses are warranted.

In order to assess site-specific liquefaction potential for individual structures or roadway sections along the I-15 corridor, procedures based on CPT data or SPT blowcounts may be used. Utilization of CPT-based criteria for assessing liquefaction resistance are recommended over SPT based assessments for the following reasons:

1. CPT soundings provide continuous subsurface profiles rather than intermittent samples.
2. CPT data is much less sensitive to operational procedures than SPT data.
3. The data base for assessing liquefaction resistance has grown to the extent that liquefaction resistance can now be directly determined from CPT data without converting to equivalent standard penetration resistance.

Since soil behavior type classifications charts based on CPT data may not adequately portray actual subsurface conditions for each site, it is recommended that CPT data be supplemented by confirmatory borings from which relatively undisturbed samples and SPT blowcounts are obtained. Liquefaction assessments should include laboratory test results that provide soil classifications and fines content. CPT-based assessments should also be checked against occasional SPT-based assessments.

Procedures for assessing liquefaction resistance from both CPT and SPT data are subsequently presented.

7.1 CPT-Based Liquefaction Resistance Assessment

Steps for calculation of factor of safety against liquefaction using CPT data are presented in papers by Robertson, et al. (1992) and Robertson and Fear (1995). Steps are subsequently summarized.

1. Determine the design peak horizontal acceleration (a_{max}) and the design earthquake magnitude (M) for each site assessed. Recommended peak horizontal acceleration and design earthquake values are presented in the Seismic Hazard Analysis sections of this report. Peak horizontal acceleration values were generally on the order of 0.55g to 0.65g for an earthquake magnitude of 7.0.

   UDOT policy for selecting a_{max} for liquefaction assessment is outlined in Table 7.1

Table 7.1. UDOT Policy for Selection of a_{max}

Structure	Recommended a_{max} value (UDOT Policy)
Bridges and associated components	10% probability of exceedence in 250 years
Abutment embankments and/or walls that will affect the bridge integrity	10% probability of exceedence in 250 years
All other walls	10% probability of exceedence in 50 years
Embankments where damage to adjacent structures outside of the right-of-way would cost more than mitigating embankment failure	10% probability of exceedence in 50 years
All other locations	no earthquake analysis or mitigation design required

2. Develop a detailed soil profile using site specific CPT soundings supplemented by boring information and laboratory tests on samples obtained from confirmatory borings. The profile must include the depth to the water table. The profile should rely heavily on laboratory testing, however, a quick assessment of soils that may potentially be liquefiable can be made using Figure 7.1 (after Robertson, 1990). Soils susceptible to liquefaction generally fall within the area designated as Zone A on Figure 7.1. Soils that are outside of Zone A have higher friction ratios and tend to have a higher resistance to cyclic loading. However, the size of the earthquake will control the liquefaction susceptibility.

   Thin sand layers generally less than 1450 mm (4.75 feet) thick deposited between soft clay layers are often incorrectly classified as silty sand based on the CPT soil behavior type charts. The cone will start to sense the sand layers before they are actually encountered and will sense the underlying clay even when the cone has entered the sand layer. As a result, the CPT will not always measure the correct mechanical properties in thinly bedded soils. An improvement in soil classification and liquefaction assessment can be achieved if the measured cone resistance (q_c) is first corrected for layer thickness before applying the classification charts. Robertson and Fear (1995) suggest a correction factor for cone resistance (K_c) as a function of layer thickness as indicated on Figure 7.2. The correction factor should only be applied to thin sand layers embedded in thick fine grained layers. Since the trends on Figure 7.2 are relatively large, Robertson and Fear (1995) recommend a correction (corresponding to q_{cA}/q_{cB}=2) factor to be added to the measured cone resistance (q_c) as determined from Equation 7.1.

   $$K_c = 0.5\,(H/1000 - 1.45)^2 + 1.0 \qquad (7.1)$$

   where H = layer thickness in mm.

3. Calculate the cyclic shear stress ratio generated by the design earthquake (CSR) for potentially liquefiable zones identified in step 2. The CSR is determined from Equation 7.2.

   $$CSR = \tau_{av}/\sigma'_o = 0.65\,(a_{max}/g)(\sigma_o/\sigma_o')r_d \qquad (7.2)$$

   where
   a_{max} = peak horizontal acceleration at ground surface
   σ_o = total overburden pressure at potentially liquefiable layer
   σ_o' = effective overburden pressure at potentially liquefiable layer
   g = gravitational constant
   r_d = a stress reduction factor varying from a value of 1 at the ground surface to a value of 0.9 at a depth of approximately 10 meters (30 feet).

It is recommended that r_d be determined from Figure 7.3 (Seed & Idriss, 1982) using the "average values" line. For depths below 10 meters (30 feet) extrapolation of the "average values" line should be performed with judgement.

4. Normalize the cone resistance (q_c) for overburden pressure using Equation 7.3.

$$q_{c1N} = (q_c / P_a)\,(Pa/\sigma_{vo}')^{0.5} \tag{7.3}$$

where
q_{c1N} = normalized cone penetration resistance
P_a = atmospheric pressure (typically 100 kPa) in units as σ_{vo}'
σ_{vo}' = effective vertical overburden stress
q_c = cone resistance (previously corrected for thin layers)

5. Correct q_{c1N} for soils with fines contents (FC) greater than 5% to obtain equivalent clean sand normalized penetration resistance $(q_{c1N})_{cs}$. The fines content correction (Δq_{c1N}) recommended by Robertson and Fear (1995) can be determined by Equation 7.4.

$$\begin{aligned} \Delta q_{c1N} &= 60 \quad \text{if FC} \geq 35\% \\ \Delta q_{c1N} &= 0 \quad \text{if FC} \leq 5\% \\ \Delta q_{c1N} &= 2(\text{FC}-5) \text{ if } 5\% < \text{FC} < 35\% \end{aligned} \tag{7.4}$$

The fines content should be determined from laboratory testing of samples obtained from confirmatory borings. In the absence of laboratory testing, the fines content (FC) may be approximated by determining the soil behavior type from Figure 7.1 and estimating the fines content (FC) from Figure 7.4 (after Robertson & Fear, 1995). Δq_{c1N} can then be determined from Figure 7.5 or Equation 7.4.

Alternatively, it is possible to estimate fines content directly from CPT results using a soil behavior type index (I_c) based on the CPT chart by Robertson (1990). The soil behavior type index (I_c) can be determined from Equation 7.4.

$$I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{.5} \tag{7.5}$$

Where
Q = normalized penetration resistance (dimensionless)
= $(q_c - \sigma_{vo}) / \sigma_{vo}'$
F = normalized friction ratio, in percent.
= $[f_s / (q_c - \sigma_{vo})] \times 100\%$
f_s = CPT sleeve friction stress
σ_{vo}' = effective overburden stress
σ_{vo} = total overburden stress

The boundaries of soil behavior type in terms of I_c are presented in Table 7.2.

Table 7.2. Boundaries of Soil Behavior Type

Soil Behavior Type Index I_c	Zone	Soil Behavior Type (see Figure 7.1)
$I_c < 1.31$	7	Gravelly Sand
$1.31 < I_c < 2.05$	6	Sands: clean sand to silty sand
$2.05 < I_c < 2.60$	5	Sand Mixtures: silty sand to sandy silt
$2.60 < I_c < 2.95$	4	Silt Mixtures: clayey silt to silty clay
$2.95 < I_c < 3.60$	3	Clays
$I_c > 3.60$	2	Organic soils: peats

(after Robertson and Fear, 1995)

The fines content (FC) can then be calculated from Equation 7.6

Fines Content, FC (%) = $1.75I_c^3$-3.7 (7.6)
where I_c = Soil Behavior Type Index as determined from Equation 7.5.

Δq_{c1N} can then be determined from Figure 7.5 or Equation 7.6.

Some clayey soils are vulnerable to significant loss of strength due to pore pressure buildup during seismic events. Clayey soils that may be susceptible to significant strength loss have the following characteristics (Seed and Idriss, 1982):

Percent finer than 0.005 mm < 15%
Liquid limit (LL) < 35
Water content > 0.9 LL

If the clay content (determined by 0.005 mm) >20%, consider the soil non-liquefiable unless it is extremely sensitive. If the water content of any clayey soil < 0.9LL consider the soil non-liquefiable.

6. With the fines content correction (Δq_{c1N}) added to the normalized cone penetration resistance (q_N) Figure 7.6 can be used to determine the cyclic resistance ratio (CRR). When $(q_{c1N})_{cs}$ exceeds 160, liquefaction is not expected. CRR may also be determined from Equation 7.7.

$$CRR = 93\left(\frac{(q_{c1N})_{cs}}{1000}\right)^3 + 0.08 \tag{7.7}$$

where $(q_{c1N})_{cs}$ is in the range of 30 <($q_{N\ cs}$) < 160

7. If the cyclic shear stress ratio (CSR) is greater than CRR, liquefaction is likely to occur. A factor of safety against liquefaction can be determined by dividing the cyclic resistance ratio (CRR) of the soil by the cyclic shear stress ratio generated by the earthquake (CSR). The factor of safety (FS) against liquefaction is determined from Equation 7.8. A minimum safety factor of 1.1 against liquefaction is recommended. Additional analyses are recommended is FS <1.1 to determine whether a mitigation program should be developed.

$$FS=(CRR_{7.5}/CSR)MSF \tag{7.8}$$

where $CRR_{7.5}$ = cyclic resistance ratio for a magnitude 7.5 earthquake
MSF = Magnitude scaling factor FS=CRR/CSR
CSR = Cyclic shear stress ratio generated by the earthquake

For earthquake magnitudes other than 7.5, a magnitude scaling factor (MSF) should be applied to Equation 7.8. The magnitude scaling factor (MSF) can be determined from Table 7.3.

Table 7.3. Magnitude Scaling Factors

	Idriss (in press)	Youd and Noble (in press)		
Magnitude (Mw)		Probability, P_L < 20%	Probability, P_L < 32%	Probability, P_L < 50%
5.5	2.20	2.86	3.42	4.44
6.0	1.76	1.93	2.35	2.92
6.5	1.44	1.34	1.66	1.99
7.0	1.19	1.00	1.20	1.39
7.5	1.00			1.00
8.0	0.84			0.73?
8.5	0.72			0.56?

(After Youd and Noble, in press)

The values in Table 7.3 are derived from the formulas in Table 7.4.

Table 7.4. Equations for Calculating Magnitude Scaling Factors

Author	Formula	Magnitude
Idriss (in press)	$MSF = 10^{2.24}/M^{2.56}$	
Youd and Noble (in press)	$MSF = 10^{3.81}/M^{4.53}$	For M < 7
	$MSF = 10^{3.74}/M^{4.33}$	For M < 7
	$MSF = 10^{4.21}/M^{4.81}$	For M < 7.75

(After Youd and Noble, in press)

At the 1996 National Conference on Earthquake Engineering Research (NCEER) workshop on Evaluation of Liquefaction Resistance of Soils, the consensus of the participants was to recommend that for earthquakes with magnitudes less than 7.5, the lower bound of scaling factors used for practice should be those developed by Idriss (in press). The upper bound should be those developed by Youd and Noble (in press).

A recommended flowchart of CPT liquefaction assessment is included in Figure 7.7.

An example of the recommended method to assess liquefaction potential with CPT data is shown on Figures 7.8A and 7.8B for a CPT sounding completed at Interstate I-80 and 40th West. The measured cone resistance

(q_c) is corrected for thin layers and normalized to one atmosphere. The fines content (FC) is then calculated from the soil behavior type index (I_c) and the fines content correction (Δq_{c1N}) is added to the normalized cone resistance to produce an equivalent clean sand normalized penetration resistance $(q_{c1N})_{cs}$. CSR for a magnitude 7.0 earthquake producing a peak horizontal ground acceleration of 0.6g is plotted against equivalent clean sand CRR values on Figure 7B. As can been seen the majority of the sand and silty sand intervals in the upper 24 meters (75 feet) of the profile would likely liquefy under the earthquake scenario.

7.2 SPT Liquefaction Assessment

Steps for calculation of the factor of safety against liquefaction using the Simplified Empirical Procedure (Seed et al., 1985), include:

1. Determination of the design peak horizontal acceleration (a_{max}) and the design earthquake magnitude (M). Same as Step 1 of the CPT based assessment.

2. Develop a detailed soil profile of the site. This can be accomplished by completing exploratory borings supplemented by laboratory testing.

3. Calculate the cyclic shear stress ratio generated by the earthquake (CSR) for potentially liquefiable zones identified in step 2. The CSR is determined from Equation 7.2.

$$CSR = \tau_{av}/\sigma'_o = 0.65\,(a_{max}/g)(\sigma_o/\sigma_o')r_d \qquad (7.2)$$

where
- a_{max} = maximum acceleration at ground surface
- σ_o = total overburden pressure at potentially liquefiable layer
- σ_o' = effective overburden pressure at potentially liquefiable layer
- g = gravitational constant
- r_d = a stress reduction factor determined from Figure 7.3.

4. Convert the STP blowcounts measured in the field (N_m) to corrected SPT blowcounts $(N_1)_{60}$ using Equation 7.9.

$$(N_1)_{60} = C_n\,(ER/60)N_m \qquad (7.9)$$

where
- C_n = Correction factor based on the effective overburden pressure. C_n is determined from Equation 7.10
- ER = energy ratio of the hammer
- N_m = Measured SPT field blowcount

$$C_n = \sqrt{\frac{1}{\sigma'_o}} \qquad (7.10)$$

where σ'_o = effective overburden pressure in tsf (1 tsf=0.1 MPa)

5. Fines content reduces the potential for liquefaction and is taken into account by adding a synthetic increment to the corrected blowcount $(N_1)_{60}$. Knowing the fines content (FC) from laboratory test data, the synthetic blowcount increment, $\Delta(N_1)_{60}$ to be added to $(N_1)_{60}$ can be determined from Figure 7.5. The correction is read from the right side ordinate.

6. With the fines content correction $\Delta(N_1)_{60}$, added to $(N_1)_{60}$. CRR is determined from Figure 7.9 for level ground conditions and magnitude 7.5 earthquakes. For earthquake magnitudes other than 7.5, a magnitude scaling factor (MSF) should be applied as indicated in step 7 of the CPT based assessment.

   Determination of the factor of safety against liquefaction can also be assessed using the same procedure as recommended in step 7 of the CPT based assessment.

The factor of safety against liquefaction was determined for several depths at the same site evaluated in the CPT based assessment example. A boring log obtained from the site is presented as Figure 7.10. Spreadsheets in Figures 7.11A and 7.11B calculate the factor of safety against liquefaction for depths of 16, 45 and 75 feet. Since the logs are in feet, the spreadsheets also use English units, however, the procedure would be the same for metric units. The results are comparable with those of the CPT assessment.

7.3 Assessing the Liquefaction Problem

Liquefaction problems are generally the result of displacement failures or settlement. Displacement failures can lead to global translation of piles or bridge structures and slump failures of embankments. Liquefaction induced displacement can lead to three types of ground failure (Youd, 1993), flow failure, ground oscillation, and lateral spreading. Flow failures form on steep slopes (greater than 6%) and are characterized by large displacements (tens of feet or more). Ground oscillation occurs on flat ground where liquefaction of deeper layers has decoupled surface soil layers allowing ground oscillations or ground waves to develop. Lateral spreading occurs primarily by horizontal displacement of surficial soil layers due to liquefaction of underlying granular deposits. Lateral spreads move down gentle slopes (usually less than 6%) or slide towards a free face such as a road cut or incised river channel. Horizontal displacements may range from a few inches to several feet.

Empirical procedures provided by Bartlett and Youd (1995) to assess lateral spreading are summarized below.

1. Perform a standard liquefaction analysis for the site using procedures previously outlined.

2. If $(N_1)_{60}$ values are equal to or more than 15, the potential for lateral displacements would be small for earthquakes with magnitudes less than 8, and no additional analyses are warranted.

3. If $(N_1)_{60}$ values less than 15 are determined, then the evaluation of ground displacement is continued using the following equations:

   For free-face conditions:

   $$\text{LOG } D_H = -16.3658 + 1.1782\, M - 0.9275 \text{ LOG } R - 0.0133\, R + 0.6572 \text{ LOG } W + 0.3483 \text{ LOG } T_{15} + 4.5270 \text{ LOG } (100-F_{15}) - 0.9224\, D50_{15}$$

   For ground slope conditions:

   $$\text{LOG } D_H = -15.7870 + 1.1782\, M - 0.9275 \text{ LOG } R - 0.0133\, R + 0.4293 \text{ LOG } S + 0.3483 \text{ LOG } T_{15} + 4.5270 \text{ LOG } (100-F_{15}) - 0.9224\, D50_{15}$$

   Where: D_H = Estimated lateral ground displacement in meters (multiply by 3.3 to convert to feet)

   $D50_{15}$ = Average mean grain size in granular layer included in T_{15}, in mm.

F_{15}	=	Average fines content for granular layers included in T_{15} , in percent.
M	=	Earthquake magnitude (moment magnitude)
R	=	Horizontal distance from the seismic energy source, in kilometers (miles multiplied by 1.9)
S	=	Ground slope, in percent.
T_{15}	=	Cumulative thickness of saturated granular layers with corrected blow counts, $(N_1)_{60}$, less than or equal to 15, in meters.
W	=	Ratio of the height (H) of the free face to the distance (L) from the base of the free face to the point in question, in percent (i.e., 100H/L).

Guidance for specific input of each of the parameters is provided in Bartlett and Youd (1995).

Settlement occurs when liquefaction and attendant pore pressure dissipation causes densification of the liquefied layer. The magnitude of settlement (volumetric strain) under flat topographic conditions can be determined from charts developed by Tokimatsu and Seed (1987) based on the average cyclic shear stress ratio induced by the earthquake and the $(N_1)_{60}$ value of the soil in question. The magnitude of settlement may result in damage to overlying structures or introduction of negative skin friction on pile foundations due to settlement of liquefiable layers underlying cohesive soils. The potential effects of negative skin friction should be evaluated in pile design.

Pile design should also include an assessment of capacity reduction due to a decrease in strength of soils under seismic loading. For liquefiable layers, residual strengths should be used in determining, vertical, uplift and lateral pile capacities under seismic loading condition. Figure 7.12 (after Seed and Harder, 1990) can be used for selection of appropriate residual strengths based on equivalent clean sand SPT blowcounts in undrained conditions. The average value of the band in Figure 7.12 is recommended for use in pile design.

If the factor of safety against liquefaction is less than 1.1 (FS<1.1) for soils underlying embankments, additional analyses are recommended. For embankments constructed on potentially liquefiable soils, stability analyses should include post-earthquake conditions. Thus, residual strength values should be used that reflect a liquefied state, however, a lateral earthquake force should not be used in the post-earthquake analysis. For a post-earthquake analysis the average residual strength from Figure 7.12 should be used for liquefiable materials.

Depending on the safety factor determined from the post-earthquake analysis, the following are recommended:

1. If FS>1.1, no additional earthquake analyses are required.
2. If 1.0<FS<1.1 a deformation analysis should be performed using either the Newmark method or the Makdisi-Seed (1978) simplification of the Newmark method to evaluate the magnitude of permanent embankment deformation.
3. If FS<1.0 mitigation measures or configuration alterations should be evaluated that result in acceptable factors of safety.

APPENDIX C - DISKS CONTAINING: TABULATED DATA AND RESULTS FOR I-15 CORRIDOR INTEGRATED ANALYSIS, COMPARISON OF RESULTS WITH PREVIOUS ANALYSIS, AND ZIPPED EXCEL SPREADSHEETS WITH INTEGRATED ANALYSIS